

FOUNDATION INVESTIGATION
AND DESIGN REPORT

PROPOSED REPLACEMENT/EXTENSION OF
STRUCTURAL CULVERT 8-468C
HIGHWAY 26 FROM MEAFORD TO THORNBURY

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



I.E.
Group

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TABLE OF CONTENTS

PART A – FOUNDATION INVESTIGATION	1
1.0 INTRODUCTION	1
2.0 SITE DESCRIPTION	2
2.1 Site Location.....	2
2.2 Physiography and Topography	2
3.0 INVESTIGATION PROCEDURES	3
3.1 Field Investigation	3
3.2 Laboratory Analysis	4
4.0 SUBSURFACE CONDITIONS	4
4.1 General Subsurface Conditions	4
4.1.1 Pavement, Fill, Topsoil	5
4.1.2 Sand and Silt	5
4.1.3 Silty Clay Till.....	6
4.1.4 Shale Till Complex	6
4.2 Groundwater Conditions.....	7
PART B – FOUNDATION DESIGN.....	8
5.0 DISCUSSION AND RECOMMENDATIONS.....	8
5.1 General.....	8
5.2 Closed Box Culvert	9
5.3 Open Footing Culvert (Spread Footing Foundations)	10
5.4 Lateral Earth Pressures	10
5.5 Embankment Widening	12
5.6 Excavation, Groundwater Control and Temporary Shoring.....	13
5.7 Frost Protection.....	14
5.8 Scour Depth	14
6.0 STATEMENT OF LIMITATION	15

Drawings & Appendices

Drawing 1	Borehole Locations and Soil Strata	
Appendix “A”	Explanation of Terms Used in Report	
	Record of Borehole Sheets	Boreholes 03A-1 to 03A-3
Appendix “B”	Laboratory Test Results	
	Grain Size Distribution	Figures 1, 3, 4 and 6
	Plasticity Chart	Figures 2, 5 and 7
Appendix “C”	Limitations of Report	
Appendix “D”	Site Photographs	

PART A – FOUNDATION INVESTIGATION

1.0 INTRODUCTION

This report presents the results of a foundation investigation carried out in July and August 2007 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 replacement/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 was 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. This report covers the site of Structure 8-468C.

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 wing walls will be required at the inlet of Structure 8-468C in accordance with the RFP terms of reference. Partial or full replacement of the culvert may be required pending on the results of the culvert inspection specified under Section 6.3.1 of the RFP document. Since preparation of the draft foundation investigation and design report, the final culvert recommendations indicate culvert repairs only with no replacement. This final report is completed to provide foundation data for future use.

Authorization to complete this assignment was given by Mr. Dan Green, 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

Structure 8-468C is located on Highway 26, approximately 0.7 km east of the east limit of the Town of Meaford, located at Station 23+876. Photographs of this culvert site are presented in Appendix "D". The existing structure is a reinforced concrete, non-rigid frame open footing culvert with a span of 3.05 meters, a height of 1.52 meters and a length of 22.6 m, with an overfill height of approximately 0.5 m. The culvert opening dimensions were provided in the RFP documents.

The culvert site is located within a drainage valley in which the stream flows northerly. The approach embankments were built on both the east and west sides of the culvert, with a maximum height of approximately 2.5 m. The embankment slopes are typically 2.5H to 3H:1V and are grass covered. No signs of embankment slope instability were observed at the time of this foundation investigation.

The headwalls that exist at both ends of the culvert are constructed of gabion baskets. Brown silt to silty clay deposit was noted at the streambed. There was approximately 0.5 m of water running in the creek.

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 asphalt pavement surface over the existing culvert is near elevation 217.7 m while the ground surface at the base of the embankment and in the flood plain is near elevation 215.5 m.

3.0 INVESTIGATION PROCEDURES

3.1 Field Investigation

Between July 30 and August 1, 2007, a CME 55 drill rig was supplied by London Soil Test Limited and used on site for drilling and Standard Penetration Testing (SPT, following the procedures of ASTM D 1586). Three (3) boreholes were drilled and sampled to obtain data for foundation design of the proposed rehabilitation work and potential culvert replacement. The locations of the boreholes are shown on Drawing 1.

The culvert borehole numbering system was established from the catchment area numbering system used in the Drainage Report of this project, as agreed with Stantec. For culverts numbered 1 to 9 inclusive, a preceding "0" was added to facilitate organization of laboratory data, and preparation of the borehole logs and laboratory reports. A letter "A" or "B" was also added after the culvert numbers to delineate Part A or Part B of this assignment. The boreholes were numbered 03A-1 to 03A-3 for the subject culvert and the depths of sampling were as follows:

Borehole No.	Depth of Sampling (m)
03A-1	4.72
03A-2	5.49
03A-3	3.96

The boreholes were drilled using continuous flight solid 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. It is noted that the measured shear strength value would be slightly lower than the actual value due to sampling disturbance.

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

The stations, offsets and ground surface elevations at the as drilled borehole locations were surveyed by AGM London 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, Atterberg Limit tests and unit weight tests were performed on selected samples.

The results of the laboratory testing are presented on the Record of Borehole sheets (Appendix "A"), and Laboratory Test Results (Figures 1 to 7, 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.

In general, the subsurface deposits at the site consist of compact embankment fill placed on a thin layer of very stiff to hard silty clay till which is in turn underlain by a hard shale till complex.

4.1.1 Pavement, Fill, Topsoil

Borehole 03A-2, which was located at the north edge of existing pavement in the shoulder area, encountered 300 mm shoulder gravel. Underlying the shoulder gravel is the embankment fill material that extended to a depth of 1.52 m (elevation 216.13 m). The fill beneath the shoulder gravel consists of silty clay with embedded sand and gravel.

At Boreholes 03A-1 and 03A-3, topsoil was contacted to depths of 0.25 m (elevation 217.28 m) and 0.20 m (elevation 215.34 m) respectively.

The topsoil in Borehole 03A-3 is underlain by fill materials extending to a depth of 1.37 m (elevation 214.17 m). A single grain size distribution of the embankment fill is shown on Figure 1 of Appendix “B”.

Standard penetration tests yielded “N”-values from 14 to 20 blows per 0.3 m. This fill is brown in colour and the measured natural moisture contents range from 4 to 21%.

One (1) sample was tested and the results are shown in Figure 2 of Appendix “B” and summarized below:

Liquid Limit (W_L)	41%
Plastic Limit (W_P)	24%
Plasticity Index (I_p)	21%

Unit weight of the fill was not determined due to the disturbance of the soil samples during sampling and sample retrieval.

Based on the above field and laboratory test results, together with visual and tactile examination, the fill beneath the shoulder gravel consists of clayey soil of medium plasticity and in a compact compactness condition.

4.1.2 Sand and Silt

The topsoil at Borehole 03A-1 was underlain by a 0.97 m thick sand and silt layer which extended to a depth of 1.22 m (elevation 216.31 m). One (1) grain size analysis was performed and the results are plotted on Figure 3 of Appendix “B”.

Standard penetration testing yielded an “N”-value of 24 blows per 0.3 m. The natural moisture content was measured at 12%. Based on the above field and laboratory test results, together with visual and tactile examination, the sand and silt deposit is considered to have compact compactness condition with a moist moisture condition.

4.1.3 Silty Clay Till

A stratum of grey silty clay till was contacted below the fill materials at Boreholes 03A-2 and 03A-3 and the sand and silt layer at Borehole 03A-1, and extended to depths of 2.13 to 2.44 m below the ground surface. Three (3) grain size analyses were performed on the silty clay till deposit and the results are presented on Figure 4 of Appendix "B".

Standard penetration tests yielded "N"-values from 17 to 30 blows per 0.3 m. Three (3) samples were tested and exhibited the following Atterberg Limits. These results are shown in Figure 5 of Appendix "B" and summarized below:

Liquid Limit (W_L)	36 to 37%, average at 36.7%
Plastic Limit (W_P)	21 to 23%, average at 22.0%
Plasticity Index (I_P)	14 to 15%, average at 14.7%

The natural moisture contents were in the range of 10 to 19%. These results are characteristic of clayey soils of medium plasticity (CI). The measured natural moisture contents are near or below the measured plastic limits and indicate that the deposit is pre-consolidated.

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

4.1.4 Shale Till Complex

The silty clay till was underlain by a stratum of grey shale till complex, and extended to the full depth of each borehole. One (1) grain size analysis was performed on the shale till complex and the results are presented on Figure 6 of Appendix "B".

Standard penetration tests yielded "N"-values over 100 blows per 0.3 m. A single unit weight was measured on the shale till complex and yielded a result of 23.2 kN/m³.

One (1) sample was tested and exhibited the following Atterberg Limit. These results are shown in Figure 7 of Appendix "B" and summarized below:

Liquid Limit (W_L)	39%
Plastic Limit (W_P)	22%
Plasticity Index (I_P)	17%

The natural moisture contents were in the range of 7 to 9%. These results are characteristic of clayey soils of medium plasticity (CI). The measured natural moisture contents are below the measured plastic limits and indicate that the deposit is pre-consolidated. Based on the above

field and laboratory test results, together with visual and tactile examination, the shale till complex exhibited hard consistency.

4.2 Groundwater Conditions

The groundwater condition was monitored during and upon completion of sampling. On completion of drilling, free groundwater was not observed in all three boreholes.

The water level in the creek was approximately 0.5m above the creek bottom at the time of the investigation and reflected a low flow condition.

It should be noted that the groundwater level will fluctuate seasonally and in response to weather events. Under adverse conditions, water could be perched within the embankment fill and sand and silt soils, and on top of the silty clay till. It is reasonable to assume that groundwater could be similar to the water level in the creek 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 replacement/extension of Structure 8-468C, 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.

Structure 8-468C is located on Highway 26, approximately 0.7 km east of the east limit of the Town of Meaford, located at Station 23+876. Photographs of this culvert site are presented in Appendix “D”. The existing structure is a reinforced concrete, non-rigid frame open footing culvert with a span of 3.05 meters, a height of 1.52 meters and a length of 22.6 m, with an overfill height of approximately 0.5 m. The culvert opening dimensions were provided in the RFP documents.

The culvert site is located within a drainage valley in which the stream flows northerly. The approach embankments were built on both the east and west sides of the culvert, with a maximum height of approximately 2.5 m. The embankment slopes are typically 2.5H to 3H:1V and are grass covered. No signs of embankment slope instability were observed at the time of this foundation investigation.

The headwalls that exist at both ends of the culvert are constructed of gabion baskets. Brown silt to silty clay deposit was noted at the streambed. There was approximately 0.5 m of water running in the creek.

The Culvert Inspection and Evaluation Report and PDR recommended culvert rehabilitation along with repair replacement of the gabion retaining walls, subject to Stantec’s culvert inspection. This report covers the potential replacement of the wingwalls and the potential partial or full replacement of the culvert. The replacement culvert, if required, will consist of either a precast concrete box culvert, a cast-in-place box culvert or a rigid frame open-footing culvert.

Alternatively, the replacement culvert could be constructed as a rigid frame, open-footing culvert which will be over-built to encompass the existing culvert. This alternative will allow working in the dry and removal of the existing culvert after completion of the new culvert.

It is assumed that the replacement culvert, if required, will be of similar dimensions as those of the existing culvert.

5.2 Closed Box Culvert

The soils encountered at the subject site are considered suitable for the support of a box culvert foundation. Results of all boreholes put down along the proposed culvert alignment indicate that the founding subgrade consists of very stiff to hard silty clay and shale till complex.

The 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.

Based on the borehole results, the box culvert should be designed to bear on the native, undisturbed silty clay till or shale till complex at the elevation and bearing resistances shown below:

Highest Elevation (m)	Factored Geotechnical Resistance at ULS (kPa)	Geotechnical Reaction at SLS (kPa)
214.00	600	300

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

Minor dewatering with strategically located sumps and trenches will likely be required to facilitate foundation 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. Under wet weather and or site condition, the silty clay till could easily be disturbed. In this regard, a 50 mm thick layer of lean concrete should be placed on the subgrade immediately after subgrade preparation to protect its integrity under wet conditions.

A 300 mm thick OPSS Granular "A" bedding and a 75mm thick levelling granular course as per OPSS422, or bedding as specified by the precast manufacturer should be placed on the prepared subgrade to achieve a uniform support for a precast concrete culvert. The Granular "A" layer

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), or as specified by the precast manufacturer.

5.3 Open Footing Culvert (Spread Footing Foundations)

Based on the borehole results, spread footings may be used for the culvert walls, headwalls (wingwalls) and retaining walls, and designed to bear on the undisturbed native silty clay till or shale till complex at the elevation and bearing resistances shown below:

Highest Elevation (m)	Factored Geotechnical Resistance at ULS (kPa)	Geotechnical Reaction at SLS (kPa)
214.00	600	300

The SLS value given above is based on a maximum settlement of 25 mm. This can be achieved provided that the founding subgrade is undisturbed during the construction.

Under inclined loading conditions, the bearing resistance at ULS should be reduced in accordance with Clause 6.7.4 of CAN/CSA-S6-06.

Immediately upon excavation, the exposed subgrade should be inspected and approved by the geotechnical engineer.

5.4 Lateral Earth Pressures

The lateral earth pressures acting on the culvert walls, headwalls (wing walls) and retaining walls (armour stone, 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 µm) 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, competent silty clay till or shale till complex founding soils can be calculated using an adhesion of 50 kPa. Alternatively, a coefficient of friction (friction factor) of 0.45 may be used for concrete on very stiff to hard silty clay till or shale till complex 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.

5.5 Embankment Widening

The existing approach embankments are up to 2.5 m high adjacent to the existing culvert. For the widening of the 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. 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 widening the approach embankments on the inorganic native soils are anticipated for embankment slope of 2.5H:1V and up to 2.5 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.

5.6 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 Highway 26 is between 3 and 4 m.

Excavation to depths of up to 4 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 creek can be controlled by 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.

It should be pointed out that if the founding soil is disturbed, excessive settlements could occur after structural loads are applied. The founding level will be located below the streambed and, therefore, a minimum 50 mm thick lean concrete working mat should be placed immediately after excavation and subgrade preparation for footings to protect the integrity of the bearing surface and to facilitate placement of reinforcing steel. All foundation excavations, bearing surfaces, and placement of lean concrete mat should be inspected and approved by the geotechnical engineer.

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 and compact sand and silt soils encountered at this site are classified as Type 3 soils and the very stiff to hard silty clay till soils are classified as Type 2 soils. Saturated cohesionless soils are classified as Type 4 soils.

For the Type 2 soils, the excavation shall be cut to near vertical in the bottom 1.2 m and then trimmed back to 1H:1V. Within the Type 3 soils and above the water table, 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.

Temporary support within the overfill of the existing and the new partially constructed embankment/culvert may be required to facilitate culvert construction and to maintain access for construction and local traffic, and emergency vehicles. The staging of different phases of this work should be examined to determine if roadway protection is required. Roadway protection is generally a contractor design/build item in accordance with OPSS 539, SP105S19 and current MTO practices. Geotechnical parameters for the design of temporary support structures are provided in Section 5.4.

5.7 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.

5.8 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. Silt and sand deposit and silty clay till could be exposed at the streambed, and their permissible velocities are 0.6 m/s and 1.5 m/s respectively (based on American Society of Civil Engineers publication, 1926, reprinted as Design Chart 2.17, MTO Drainage Management Manual 1997).

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Final Report
Page 15
December 17, 2008

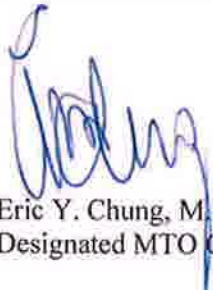
6.0 STATEMENT OF LIMITATION

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

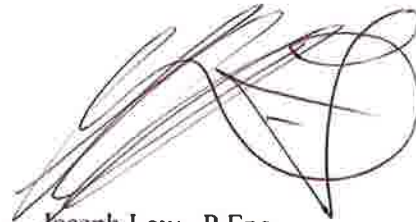
The Limitations of Report, as Quoted in Appendix "C", is an integral part of this report.

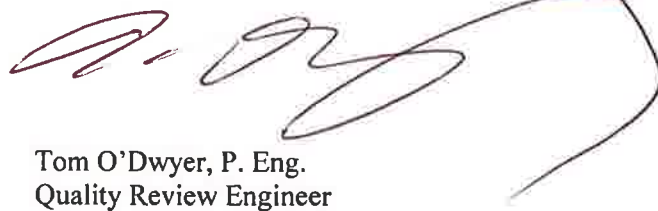
We trust that we have completed the assignment within the Terms of Reference for this project. If there are any questions concerning this report, please do not hesitate to contact our office.

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



Infrastructure Engineering Group Inc.

Ministry of Transportation/Stantec Consulting Ltd.
G.W.P. 57-00-00
Rehabilitation of Highway 26 from Meaford to Thornbury
Agreement Agreement # 3006-E-0002

07-6-IEG1-8-468C
Final Report
Drawing 1
December 17, 2008

Drawing 1
Borehole Locations
And
Soil Strata

METRIC
DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN

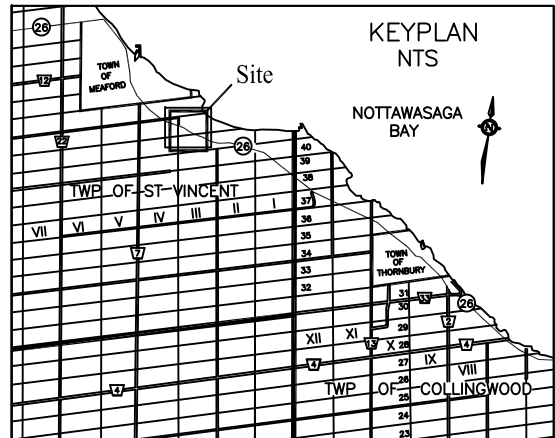
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Culvert # 8-468C
Highway 26
BORE HOLE LOCATIONS & SOIL STRATA

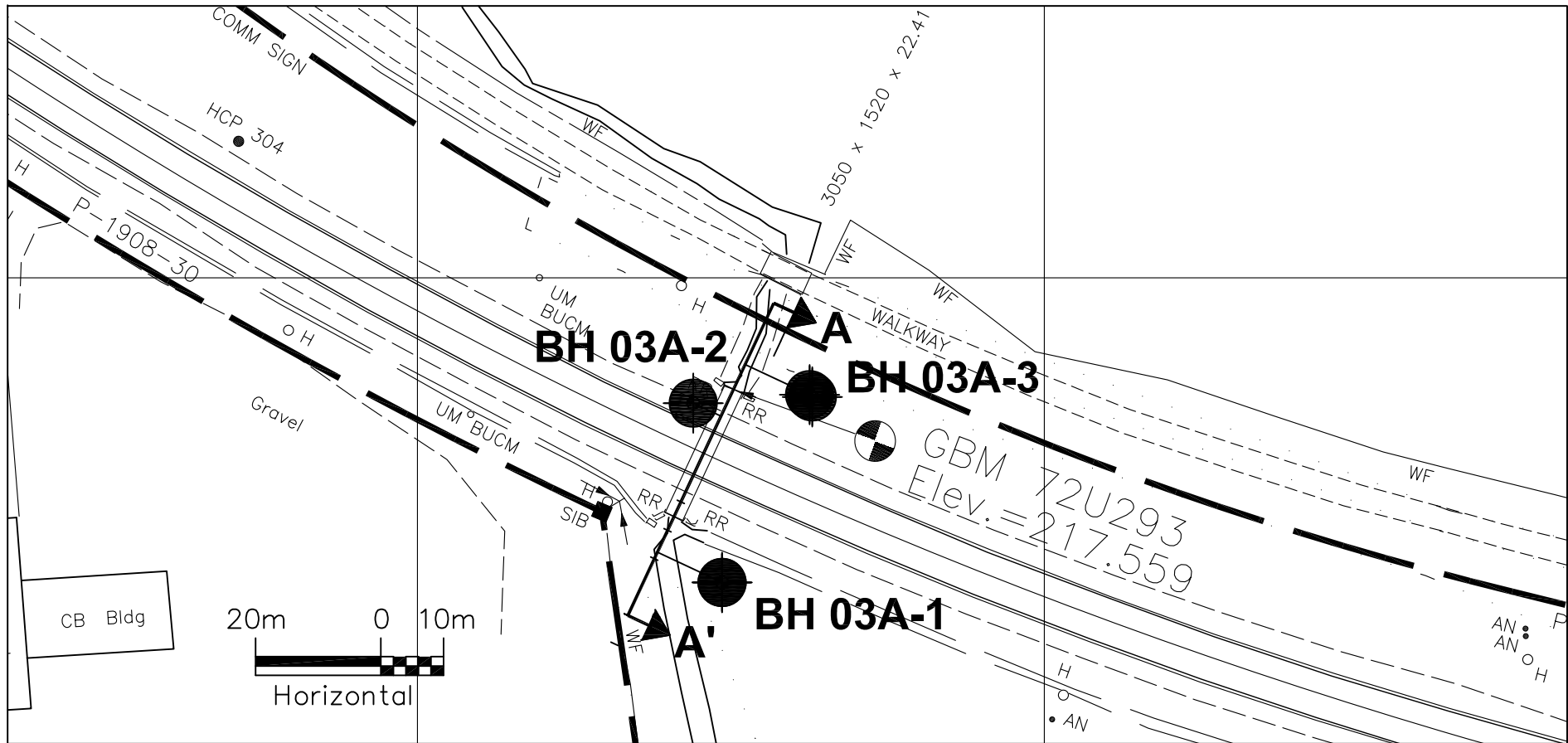
SHEET
1

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

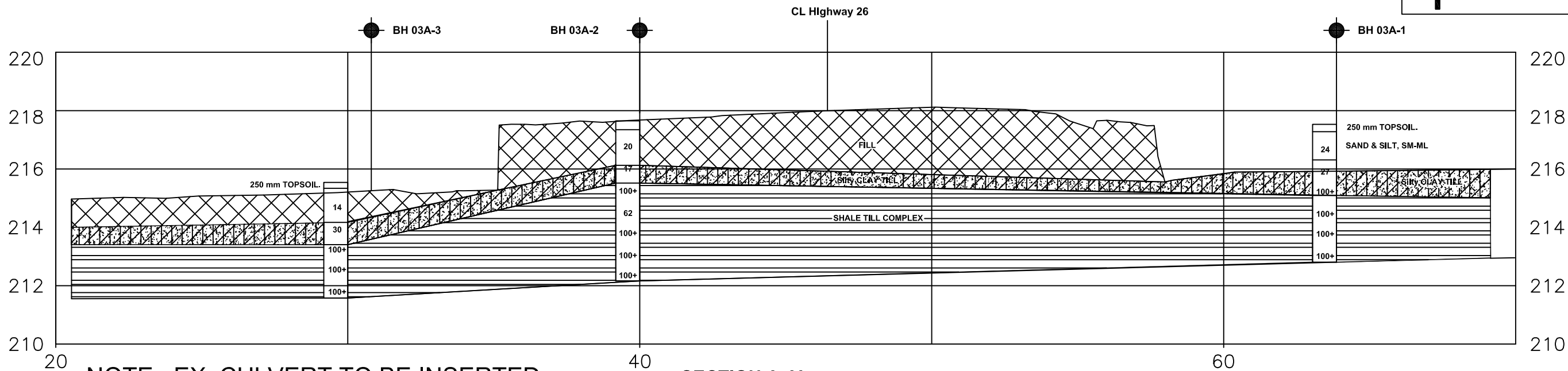


LEGEND

- Bore Hole
- Dynamic Cone Penetration Test (Cone)
- Bore Hole & Cone
- 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
- Standpipe



BOREHOLE LOCATION PLAN



SECTION A-A'
CENTERLINE OF CULVERT



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.
- SUBGRADE ELEVATION OF THE EXISTING FOOTING NOT KNOWN AND IS ESTIMATED TO BE AT 1.2m BELOW THE CREEK BED.
- 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	DATE	SITE	DIST	Owen Sound
		NORTH	EAST						
03A-1	217.53	4939651	219449	SUBM'D	J.L.	CHECKED E.C.	DATE 25/01/08	SITE	8-468C
03A-2	217.65	4939680	219444	DRAWN	J.L.	CHECKED J.L.	APPROVED E.C.	DWG	1
03A-3	215.54	4939681	219463						

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G.W.P. 57-00-00
Rehabilitation of Highway 26 from Meaford to Thornbury
Agreement # 3006-E-0002

07-6-IEG1-8-468C
Final Report
Appendix A
December 17, 2008

Appendix A

Explanation of Terms Used in Report

Record of Borehole Sheet

Boreholes 03A-1 to 03A-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_c	kPa	REMOULDED SHEAR STRENGTH
S_t	1	SENSITIVITY = $\frac{c_u}{\tau_c}$

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 03A-1

1 OF 1

METRIC

W.P. GWP 57-00-00 LOCATION Northing - 4939651, Easting - 219449 ORIGINATED BY JL
 DIST Owen Sound HWY 26 BOREHOLE TYPE S/S Augering, 110 mm dia. COMPILED BY JL
 DATUM Geodetic DATE 08.01.07 - 08.01.07 CHECKED BY EC

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	PENETR. RESISTANCE STANDARD ● DYN. CONE		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							WATER CONTENT (%)	
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE								
217.53 0.00	Ground						20 40 60 80 100									
217.28 0.25	250 mm TOPSOIL.															
	SAND & SILT, SM-ML Brown, moist, compact.		1	SPT	24									3 47 39 11 (50)		
216.31 1.22																
	Silty CLAY TILL, CI Grey, moist, very stiff, with embedded sand and gravel.		2	SPT	27					225				2 5 58 34 (92)		
215.09 2.44			3	SPT	100+											
	SHALE TILL COMPLEX Grey, moist, hard.		4	SPT	100+											
			5	SPT	100+											
212.81 4.72	End of borehole.		6	SPT	100+									Spoon and auger refusal. Borehole dry and open @ completion.		

JOE MTO 07-6-IEG1.GPJ ONTARIO.MOT.GDT 12/15/08

+ 3, X 3: Numbers refer to
Sensitivity

○ 150 UNCONFINED SHEAR STRENGTH INFERRED FROM POCKET PENETROMETER READINGS

RECORD OF BOREHOLE No 03A-2

1 OF 1

METRIC

W.P. GWP 57-00-00 LOCATION Northing - 4939680, Easting - 219444 ORIGINATED BY JL
 DIST Owen Sound HWY 26 BOREHOLE TYPE S/S Augering, 110 mm dia. COMPILED BY JL
 DATUM Geodetic DATE 07.30.07 - 08.01.07 CHECKED BY EC

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	PENETR. RESISTANCE STANDARD ● DYN. CONE		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							WATER CONTENT (%)	
								○ UNCONFINED ● QUICK TRIAXIAL	+ FIELD VANE × LAB VANE							
217.65 0.00	Ground							20	40	60	80	100	10	20	30	GR SA SI CL
217.35 0.30	300 mm sand and gravel FILL.															
	FILL Brown to grey, moist, compact, consisting of silty clay with embedded sand and gravel.		1	SPT	20											
216.13 1.52	Silty CLAY TILL, CI Grey, moist, very stiff, with embedded sand and gravel.		2	SPT	17											12 15 44 29 (73)
215.52 2.13	SHALE TILL COMPLEX Grey, moist, hard.		3	SPT	100+											
			4	SPT	62											1 2 61 36 (97)
			5	SPT	100+											
			6	SPT	100+											
			7	SPT	100+											
212.16 5.49	End of borehole.															Spoon and auger refusal. Borehole dry and open @ completion.

JOE MTO 07-6-JEG1.GPJ ONTARIO.MOT.GDT 12/15/08

+ 3, X 3: Numbers refer to
Sensitivity

○ 150 UNCONFINED SHEAR STRENGTH INFERRED FROM POCKET PENETROMETER READINGS

RECORD OF BOREHOLE No 03A-3

1 OF 1

METRIC

W.P. GWP 57-00-00 LOCATION Northing - 4939681, Easting - 219463 ORIGINATED BY JL
 DIST Owen Sound HWY 26 BOREHOLE TYPE S/S Augering, 110 mm dia. COMPILED BY JL
 DATUM Geodetic DATE 08.01.07 - 08.01.07 CHECKED BY EC

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	PENETR. RESISTANCE STANDARD ● DYN. CONE		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
								○ UNCONFINED	+ FIELD VANE						● QUICK TRIAXIAL	× LAB VANE	WATER CONTENT (%)
215.54 0.00	Ground						20	40	60	80	100	10	20	30		GR SA SI CL	
215.34 0.20	200 mm TOPSOIL.																
	FILL Grey, moist, compact, consisting of silty clay with embedded sand and gravel.		1	SPT	14									41		9 18 46 27 (73)	
214.17 1.37	Silty CLAY TILL, CI Grey, moist, very stiff to hard, with embedded sand and gravel.		2	SPT	30											3 6 60 31 (91)	
213.41 2.13	SHALE TILL COMPLEX Grey, moist, hard.		3	SPT	100+												
			4	SPT	100+												
211.58 3.96	End of borehole.		5	SPT	100+											Spoon and auger refusal. Borehole dry and open @ completion.	

JOE MTO 07-6-IEG1.GPJ ONTARIO.MOT.GDT 12/15/08

+ 3, X 3: Numbers refer to
Sensitivity

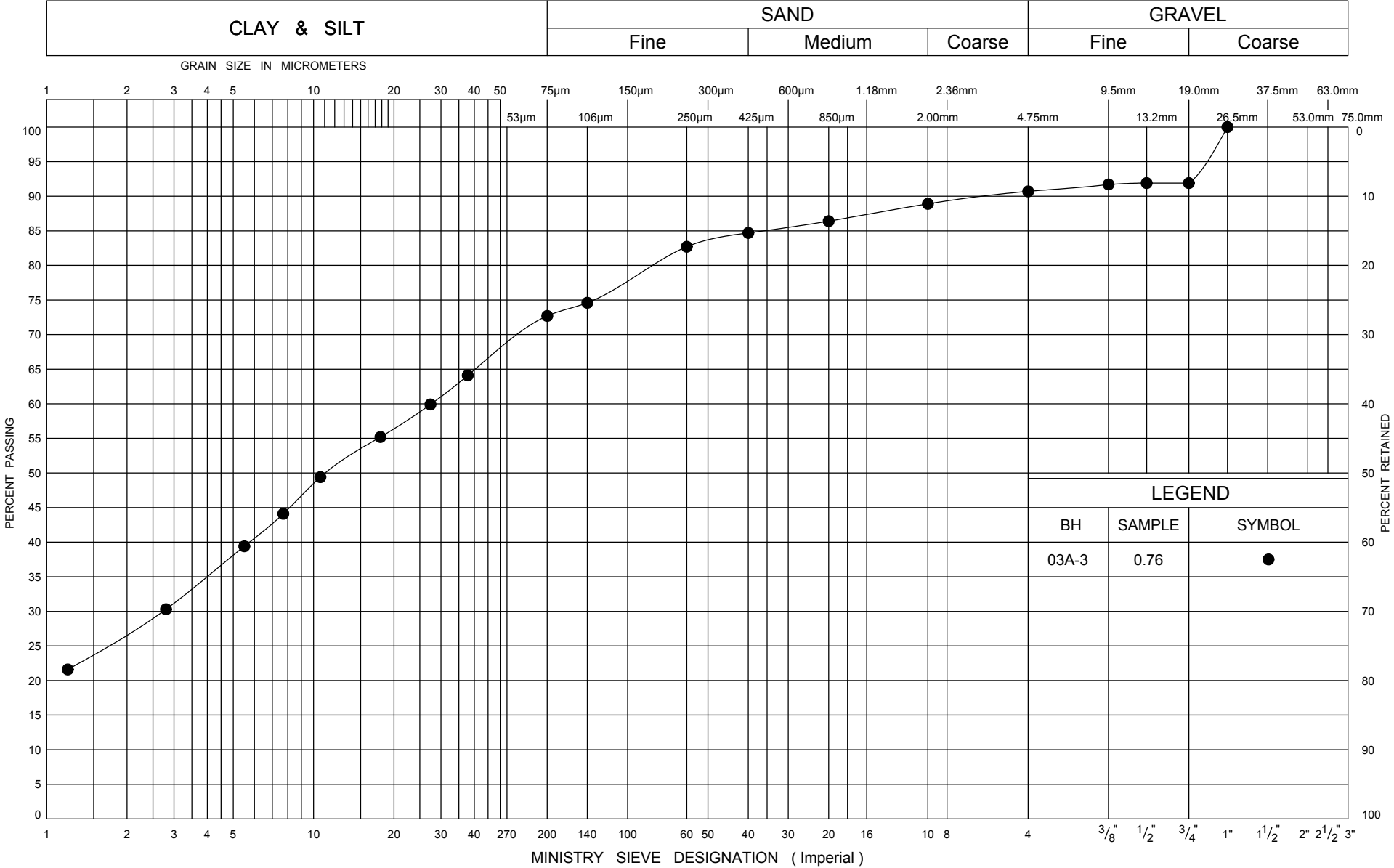
○ 150 UNCONFINED SHEAR STRENGTH INFERRED FROM POCKET PENETROMETER READINGS

Appendix B

Laboratory Test Results

Grain Size Distribution	Figures 1, 3, 4 and 6
Plasticity Chart	Figures 2, 5 and 7

UNIFIED SOIL CLASSIFICATION SYSTEM

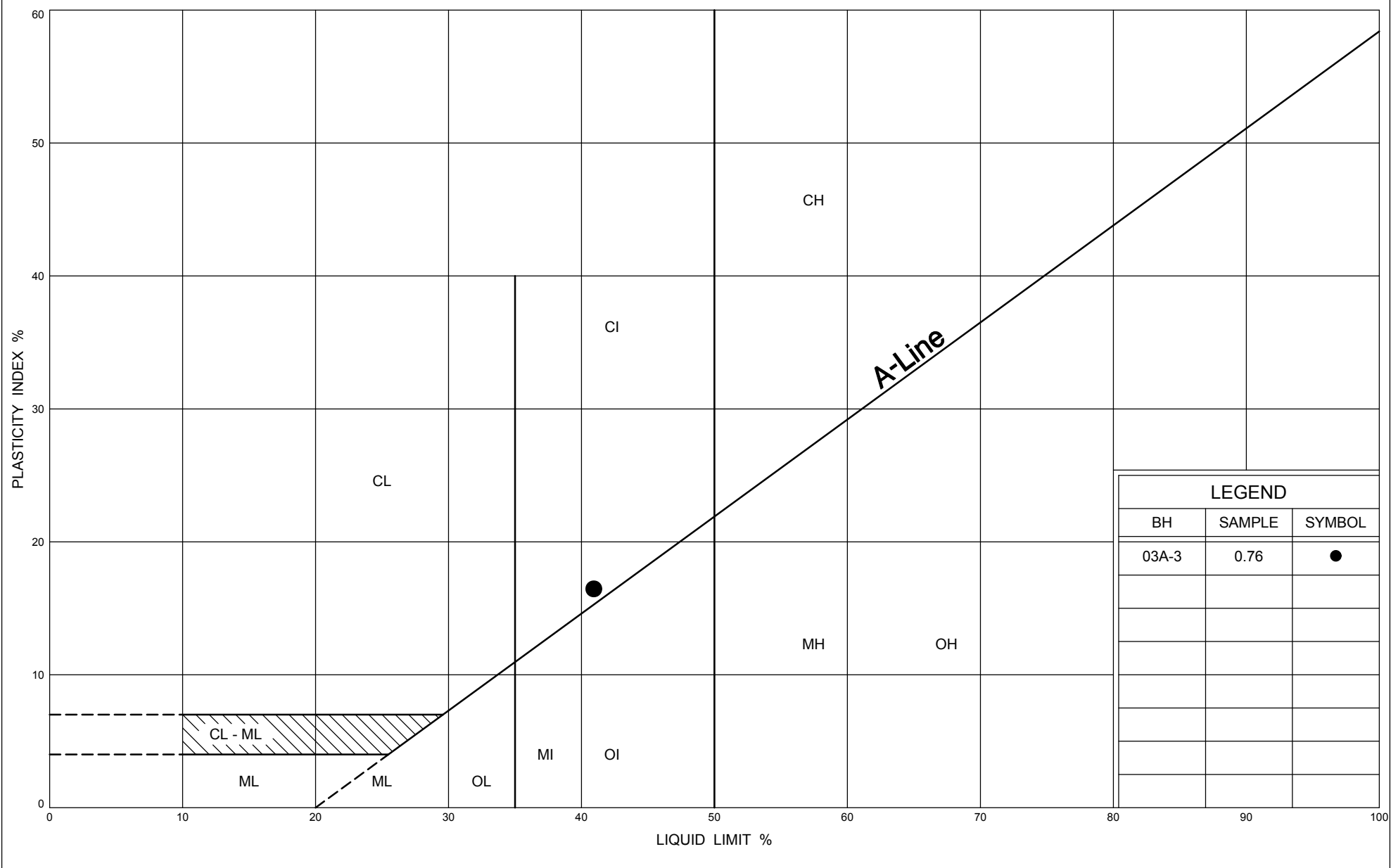


GRAIN SIZE DISTRIBUTION
FILL

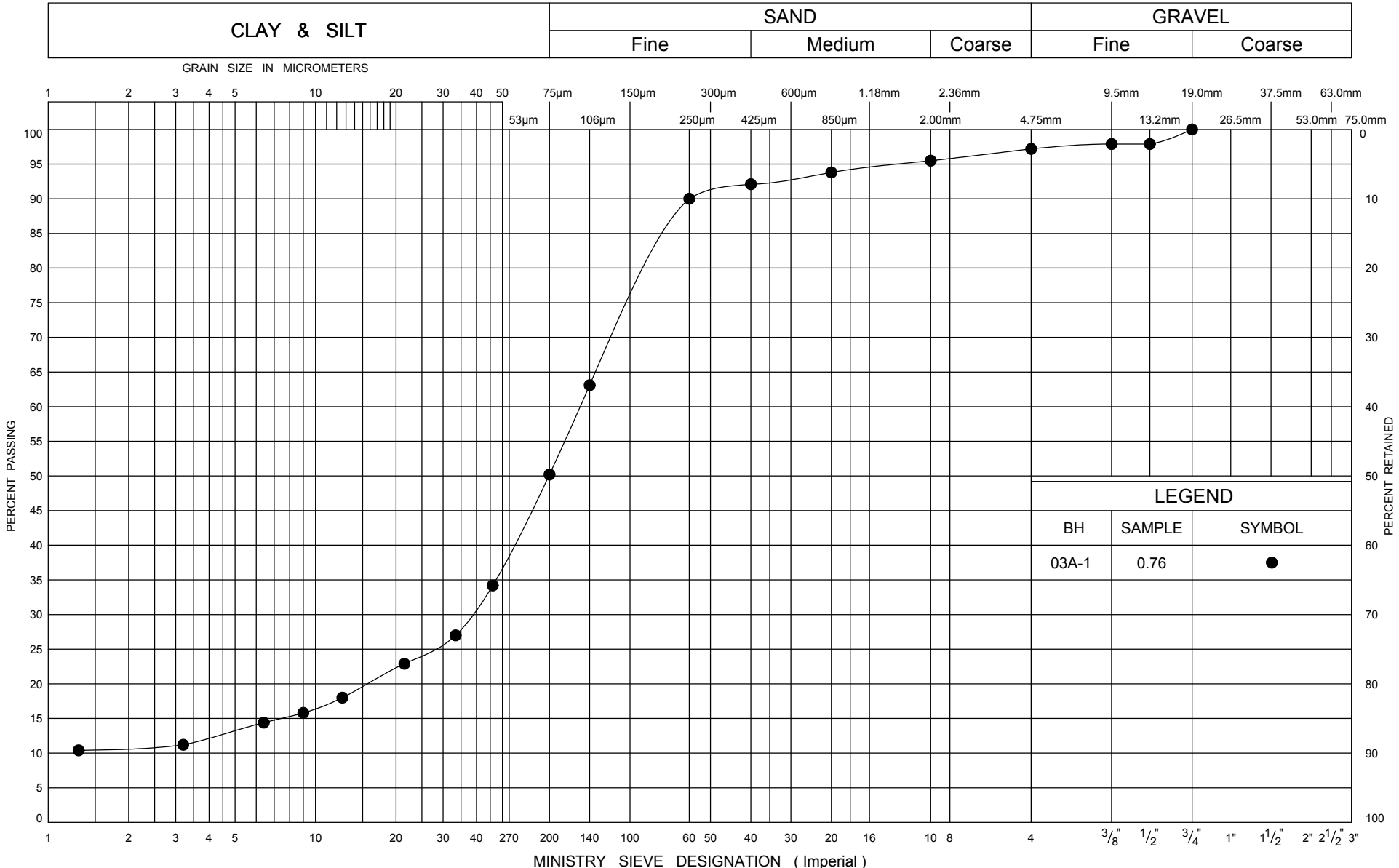
FIG No 1

GWP 57-00-00

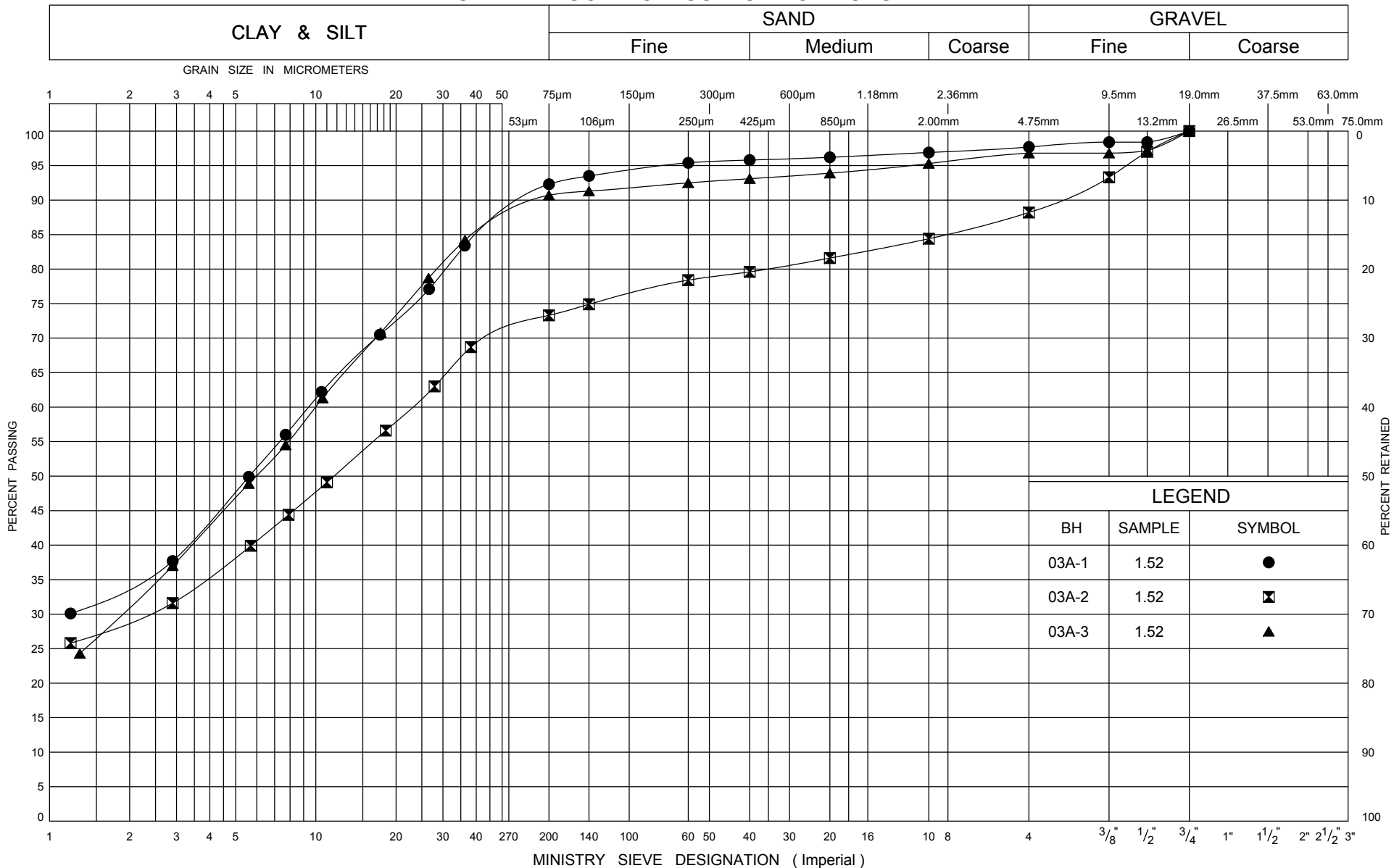
HWY 26, Thornbury to Meaford

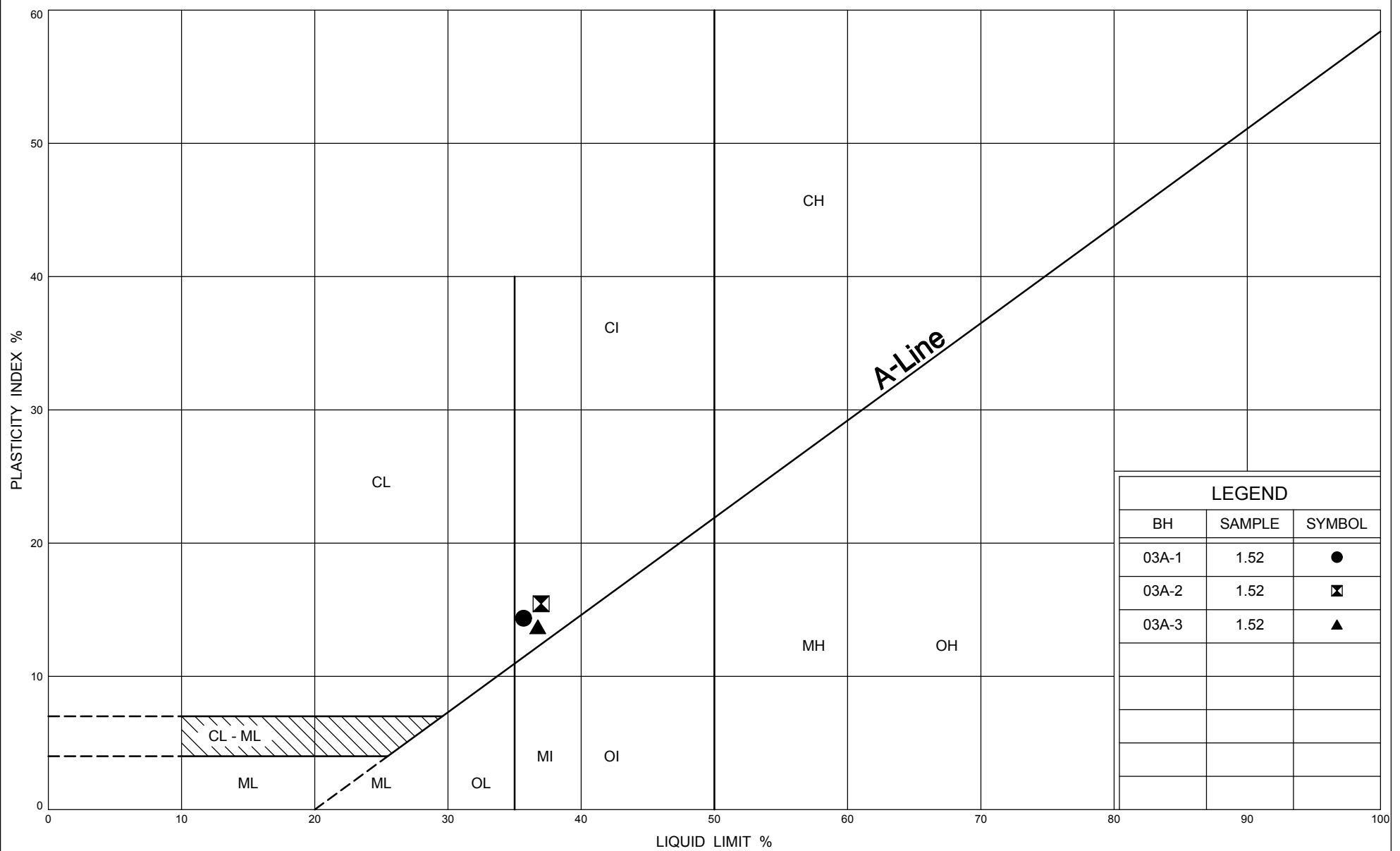


UNIFIED SOIL CLASSIFICATION SYSTEM



UNIFIED SOIL CLASSIFICATION SYSTEM





Ministry of
Transportation

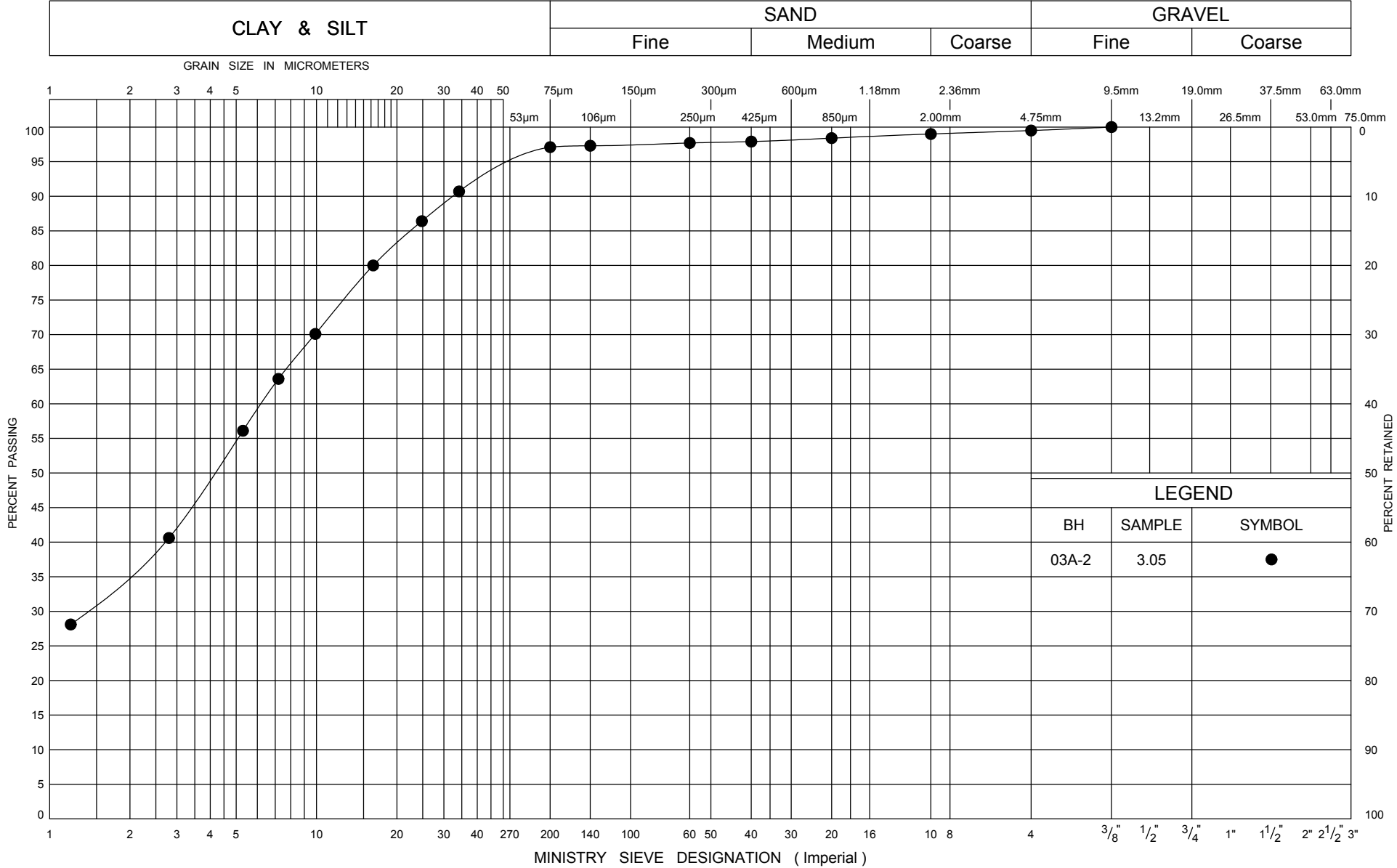
PLASTICITY CHART SILTY CLAY TILL, CI

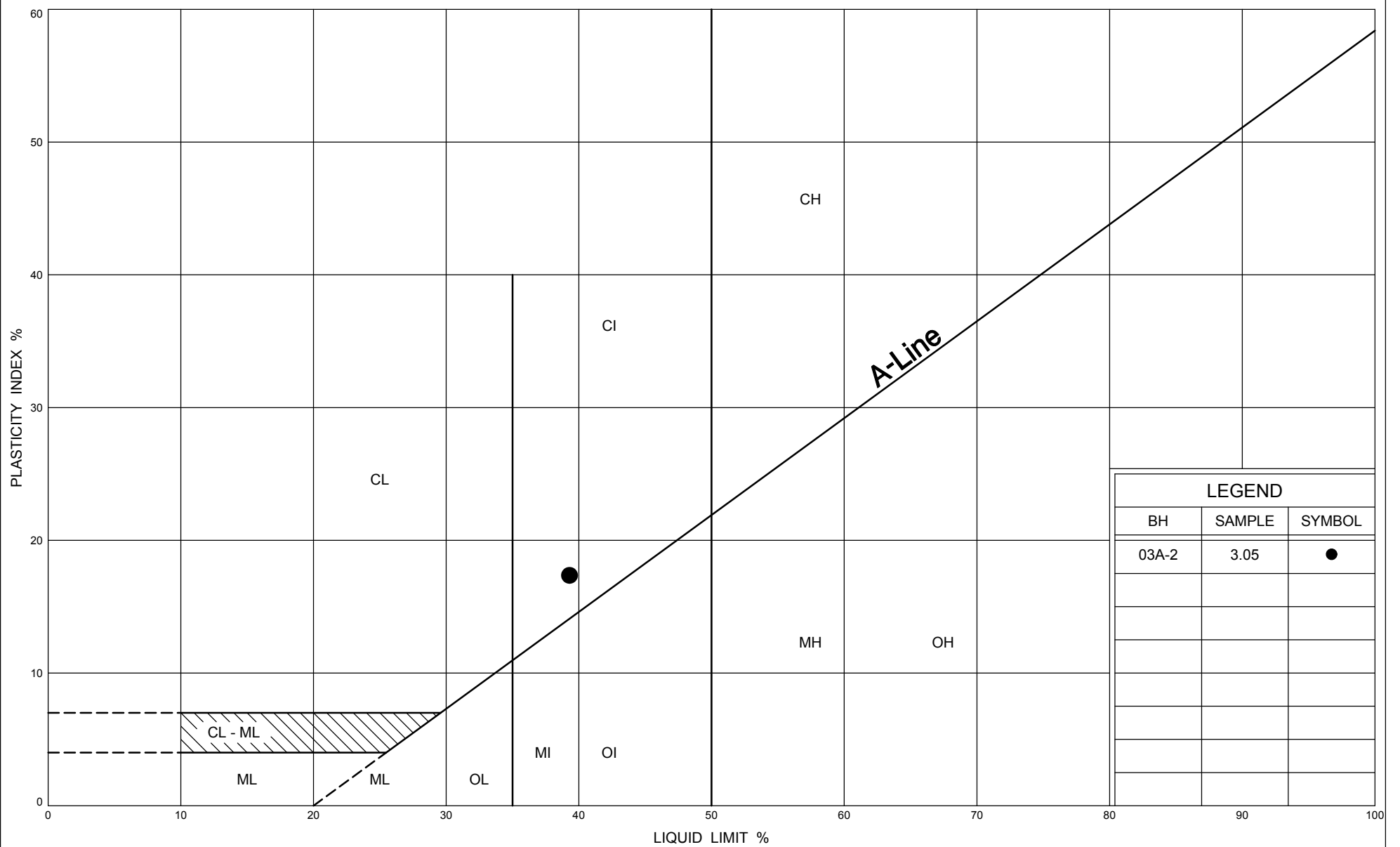
FIG No 5

GWP 57-00-00

HWY 26, Thornbury to Meaford

UNIFIED SOIL CLASSIFICATION SYSTEM





Ministry of
Transportation

PLASTICITY CHART SHALE TILL COMPLEX

FIG No 7

GWP 57-00-00

HWY 26, Thornbury to Meaford

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Rehabilitation of Highway 26 from Meaford to Thornbury
Agreement # 3006-E-0002

07-6-IEG1-8-468C
Final Report
Appendix C
December 17, 2008

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.

Ministry of Transportation/Stantec Consulting Ltd.
G.W.P. 57-00-00
Rehabilitation of Highway 26 from Meaford to Thornbury
Agreement # 3006-E-0002

07-6-IEG1-8-468C
Final Report
Appendix D
December 17, 2008

Appendix D

Site Photographs



Station 23+876 – Looking downstream (north)



Station 23+876 – Looking upstream (south)



Station 23+876 - Downstream end (north)



Station 23+876 – Upstream end (south)