

FOUNDATION INVESTIGATION
AND DESIGN REPORT
PROPOSED RECONSTRUCTION OF
STRUCTURAL CULVERT - SITE 2-476-C
HIGHWAY 21 FROM KINCARDINE TO TIVERTON
G.W.P. 408-94-00
Agreement # 3005-E-0038



I.E.
Cory

FOUNDATION INVESTIGATION
AND DESIGN REPORT
PROPOSED RECONSTRUCTION OF
STRUCTURAL CULVERT - SITE 2-476-C
HIGHWAY 21 FROM KINCARDINE TO TIVERTON

G.W.P. 408-94-00
Agreement # 3005-E-0038

Prepared for:

SNC-Lavalin Engineers & Contractors Inc.
2200 Lake Shore Boulevard West
Toronto, Ontario
Canada M8V 1A4

Mr. Bing Wong, P.Eng

Prepared by:

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

September 28, 2007
06-8-IEG2-476

Distribution:

9	Copies -	SNC-Lavalin Engineers & Contractors Inc.
3	Digital Copies -	SNC-Lavalin Engineers & Contractors Inc.
3	Copies -	Infrastructure Engineering Group Inc.

Geocres No.: 41A-191

TABLE OF CONTENTS

PART A – FOUNDATION INVESTIGATION	1
1.0 INTRODUCTION	1
2.0 SITE DESCRIPTION	1
2.1 Site Location.....	1
2.2 Physiography and Topography.....	2
3.0 INVESTIGATION PROCEDURES.....	2
3.1 Field Investigation	2
3.2 Laboratory Analysis	3
4.0 SUBSURFACE CONDITIONS	3
4.1 General Subsurface Conditions	3
4.1.1 Pavement, Fill, Topsoil	4
4.1.2 Silty Clay Till.....	4
4.1.3 Sand.....	5
4.2 Groundwater Conditions.....	5
PART B – FOUNDATION DESIGN.....	6
5.0 DISCUSSION AND RECOMMENDATIONS.....	6
5.1 General.....	6
5.2 Closed Box Culvert	6
5.3 Open Footing Culvert (Spread Footing Foundations)	7
5.4 Lateral Earth Pressures	8
5.5 Embankment Widening	10
5.6 Excavation, Groundwater Control and Temporary Shoring.....	11
5.7 Frost Protection.....	12
5.8 Scour Depth	12
6.0 STATEMENT OF LIMITATION	13

Drawings & Appendices

Drawing 1	Borehole Locations and Soil Strata
Appendix “A”	Explanation of Terms Used in Report
	Record of Borehole Sheets
	Boreholes 2-476-C1 to 2-476-C3
Appendix “B”	Laboratory Test Results
	Grain Size Distribution
	Plasticity Chart
	Figures 1, 2 and 4
	Figure 3
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 September 2006 by Infrastructure Engineering Group Inc. on behalf of SNC-Lavalin Engineers & Contractors Inc.

The assignment involves the reconstruction/rehabilitation of the pavement structure on Highway 21 from 0.63 m north of the intersection with Highway 9 (north of Kincardine) northerly to 1.2 km east of the intersection with Bruce County Road 15 (west junction in the Village of Tiverton) for 12.4 km; including pavement rehabilitation/reconstruction throughout, snow drifting/storage treatments, structural culvert replacement/rehabilitation, non-structural culvert replacement, minor intersection improvements, drainage improvements and minor electrical work.

Foundation investigation and recommendations are required for the design and construction of culvert replacements as part of the improvement of Highway 21. Five (5) structural culverts and twenty-four (24) non-structural culverts are to be investigated. This report covers the site of Structure 2-476-C.

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. The existing culvert is to be replaced with a new culvert.

Authorization to complete this assignment was given by Mr. Bing Wong, Project Manager, SNC-Lavalin Engineers and Constructors Inc., the TPM Consultant who is completing this assignment for MTO under Agreement # 3005-E-0038.

2.0 SITE DESCRIPTION

2.1 Site Location

Structure 2-476-C is located on Highway 21, approximately 11.2 km north of Highway 9, located at station 24+856. Photographs of this culvert site are presented in Appendix D. The existing structure is a reinforced concrete, rigid frame open footing culvert having dimensions of 3.60 m wide by 2.40 m high by 41.15 m long, with an overfill height of 5.2 m. The culvert opening dimensions were provided by AGM. A brown silty clay deposit was noted at the streambed.

The culvert site is located within a drainage valley in which the stream flows westward. The approach embankments were built on both the north and south sides of the culvert, with a maximum height of approximately 8.2 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.

2.2 Physiography and Topography

The site is located within the Physiographic Region known as the “Huron–slope” (Chapman and Putnam, 1984) which occupies the area east of Lake Huron between Sarnia and Tobermory. The area is characterized by a flat topography, heavy textured soil and poor drainage. The surficial deposits consist of brown, calcareous clayey tills, which contain very few cobbles and boulders. The tills are known to be underlain by grey stratified clays of lacustrine origin.

The asphalt pavement surface over the existing culvert is near elevation 234.5 m while the ground surface at the base of the embankment and in the flood plain is between elevations 226.6 m and 228.2 m.

3.0 INVESTIGATION PROCEDURES

3.1 Field Investigation

On September 25 to 27, 2006, 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 replacement culvert. The locations of the boreholes are shown on Drawing 1.

The boreholes were numbered 2-476-C-1 to 2-476-C-3 and the depths of sampling were as follows:

Borehole No.	Depth of Sampling (m)
2-476-C-1	14.17
2-476-C-2	8.08
2-476-C-3	8.08

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 to determine the undrained shear strength of the cohesive soil deposits. It should be noted that the measured shear strength value would be 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 Infrastructure Engineering Group Inc. 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 (Appendix "A") and Laboratory Test Results (Figures 1 to 4, 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 loose to compact embankment fill placed on very stiff to hard silty clay till.

4.1.1 Pavement, Fill, Topsoil

Borehole 2-476-C-1, which was located at the west edge of existing pavement in the shoulder area, encountered 460 mm shoulder gravel. Underlying the shoulder gravel is the embankment fill material that extended to a depth of 7.77 m (elevation 226.55 m). The fill beneath the shoulder gravel typically consists of silty clay with sand and gravel and with sand and gravel pockets and occasional organic inclusions.

At Boreholes 2-476-C-2 and 2-476-C-3, topsoil and silt to silty sand fill were contacted to depths of 0.51 m (elevation 227.43 m) and 1.07 m (elevation 226.43 m) respectively.

Four (4) grain size distributions of the embankment fill and one (1) grain size distribution of the shoulder gravel are shown on Figure 1 of Appendix "B". Standard penetration tests yielded "N"-values from 5 to 21 blows per 0.3 m. The unit weight of the embankment fill was measured to be in the range of 21.8 to 22.1 kN/m³. The embankment fill is brown to grey in colour and the measured natural moisture contents range from 6 to 22%.

Based on the above field and laboratory test results, together and tactile examination, the fill materials exhibited compact to loose compactness condition.

4.1.2 Silty Clay Till

A major stratum of brown to grey silty clay till was contacted below the embankment fill at Borehole 2-476-C-1 and the topsoil and fill layers at Boreholes 2-476-C-2 and 2-476-C-3. Clayey silt, sand and silt partings, seams and layers are present within the silty clay till. The silty clay till extended to the full depth of the boreholes (i.e., elevation 219.42 m at Borehole 2-476-C-3).

Seven (7) grain size analyses were performed and the results are plotted on Figure 2 of Appendix "B". Within the silty clay till, embedded sand and gravel particles were found, as well as wet sand to silt partings and layers.

Standard penetration tests yielded "N"-values from 13 to over 100 blows per 0.3 m. A localized low "N"-value of 3 blows per 0.3 m was recorded immediately below the topsoil at Borehole 2-476-C-2. Undrained shear strength as determined from field pocket penetrometer ranged from 75 to over 225 kPa. The unit weight, based on five (5) tests, was measured to be between 22.0 and 23.8 kN/m³.

Five (5) samples were tested and exhibited the following Atterberg Limits. These results are shown in Figure 3 of Appendix "B" and summarized below:

Silty Clay Till (four samples)

Liquid Limit (W_L)	21 to 31%, average at 26.8%
Plastic Limit (W_P)	13 to 16%, average at 15.0%
Plasticity Index (I_p)	8 to 15%, average at 11.8%

Clayey Silt Layer within Silty Clay Till (one sample)

Liquid Limit (W_L)	18%
Plastic Limit (W_P)	11%
Plasticity Index (I_p)	7%

The natural moisture contents were in the range of 9 to 25%. These results are characteristic of clayey soils of low plasticity (CL). The measured natural moisture contents are generally 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 typically very stiff to hard consistency.

4.1.3 Sand

A grey fine sand layer, containing some gravel and silt, was contacted at a depth of 7.47 m (elevation 220.47 m) below the silty clay till at Borehole 2-476-C-2. A grain size distribution of the sand is shown on Figure 4 of Appendix "B". A single standard penetration test yielded an "N"-value of 54 blows per 0.3 m. The measured natural moisture content is 16%. Based on the above field and laboratory test results, the sand exhibited very dense compactness condition.

4.2 Groundwater Conditions

The groundwater condition was monitored during and upon completion of sampling. On completion of drilling, groundwater levels were noted in Boreholes 2-476-C-2 and 2-476-C-3 at depths of 1.0 and 3.0 m below ground surface, corresponding to elevation 226.94 and 224.50 m. The water entered the boreholes from the upper fill and topsoil layers.

Borehole 2-476-C-1 remained dry and open throughout the sampling operations.

The water level in the creek was approximately 0.5 m above the stream bed 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 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 reconstruction of Structure 2-476-C, 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 2-476-C is located on Highway 21, approximately 11.2 km north of Highway 9, located at station 24+856. The existing structure is a reinforced concrete, rigid frame open footing culvert having dimensions of 3.60 m wide by 2.40 m high by 41.15 m long, with an overfill height of 5.2 m. The culvert opening dimensions were provided by AGM. A brown silty clay deposit was noted at the streambed.

The culvert site is located within a drainage valley in which the stream flows westward. The approach embankments were built on both the north and south sides of the culvert, with a maximum height of approximately 8.2 m. The embankment slopes are typically 2.5H:1V and are grass covered. No signs of embankment slope instability were observed at the time of this foundation investigation.

The replacement culvert will consist of either a precast concrete box culvert or 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 understood that the replacement culverts will be of similar dimensions as recommended in the Drainage and Hydrological Study Report, but lengthened in order to accommodate the pavement widening and geometric improvements.

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 till.

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, very stiff to hard silty clay till at the elevation and bearing resistances shown below:

Highest Elevation (m)	Factored Geotechnical Resistance at ULS (kPa)	Geotechnical Reaction at SLS (kPa)
226.00	800	400

The SLS value given above is based on a maximum settlement of 25 mm for up to a 6 m wide box culvert. This can be achieved provided the founding subgrade is not disturbed during construction.

As there was no hydrostatic pressure observed during borehole sampling (within the silty clay till), piping is not considered likely to occur at the founding subgrade of the culvert.

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 1.2 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 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 precast concrete culvert. The Granular "A" layer should be compacted to 98% of the material's standard Proctor maximum dry density (SPMDD). The levelling granular course for structural precast concrete culvert 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, very stiff to hard silty clay till at the elevation and bearing resistances shown below:

Highest Elevation (m)	Factored Geotechnical Resistance at ULS (kPa)	Geotechnical Reaction at SLS (kPa)
226.00	800	400

The SLS value given above is based on a maximum settlement of 25 mm for strip footings up to 3 m wide. 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.

As there was no hydrostatic pressure observed during borehole sampling (within the silty clay till), piping is not considered likely to occur at the founding subgrade of the culvert.

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 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 = surcharge 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 35
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 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 founding soil can be calculated using an adhesion of 50 kPa. Alternatively, a coefficient of friction (friction factor) of 0.4 to 0.5 may be used for concrete on stiff to hard silty clay till as per Table 24.4 CFEM 4th Edition, 2006. For 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 8.2 m high adjacent to the proposed 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.

Due to the height of the embankment fill of greater than 8m, a 1m wide bench should be provided at mid-height of the widened embankment face.

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 8.2 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 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 (wingwalls) and retaining walls. Depending on the design that is finally selected, the anticipated maximum depth of excavation below the existing grade of Highway 21 is between 7.5 and 9.0 m.

Excavation to depths of up to 9.0 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 stream can be controlled by temporary diversion or dam and pump method. Some groundwater control will be required to handle surface runoff and minor seepage, but should be readily handled by conventional sump pumping techniques.

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 (dewatering). 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 soft silty clay till soils encountered at this site are classified as Type 3 soils and the underlying very stiff to hard silty clay till soils are classified as Type 2 soils. Saturated cohesionless soil could also be present and should be 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.

Excavation will be carried out to depths of up to 9 m and temporary support within the overfill of the existing and the new partially constructed culvert will 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 carefully to determine the requirement for roadway protection, with due considerations for the wide area of excavation required to comply with OHSa requirements. Roadway protection is generally a contractor design/build item in accordance with OPSS 539 and current MTO practices.

5.7 Frost Protection

This project is located in the Owen Sound Operations District. The design frost penetration depth for this project is 1.2 m in accordance with OPSD 3090.101. All foundations and spread footings should be provided with at least 1.2 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. The permissible velocity of the silty clay till that is exposed at the streambed is 1.5 m/s (based on American Society of Civil Engineers publication, 1926, reprinted as Design Chart 2.17, MTO Drainage Management Manual 1997).

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.

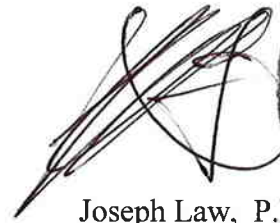
We trust that we have completed the assignment within the Terms of Reference for this project. If there are any questions concerning this report, please do not hesitate to contact our office.

Yours truly,

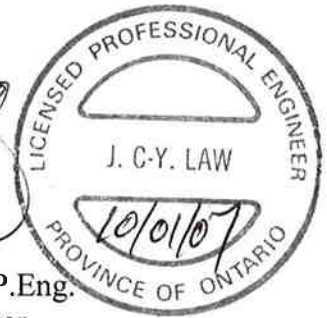
Infrastructure Engineering Group Inc.



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



Joseph Law, P.Eng.
Project Manager



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



Ministry of Transportation/SNC-LAVALIN
G.W.P. 408-94-00
Reconstruction of Highway 21 from Kincardine northerly to Tiverton
Agreement # 3005-E-0038

06-8-IEG2-476
Final Report
Drawing 1
September 28, 2007

Drawing 1
Borehole Locations
And
Soil Strata

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

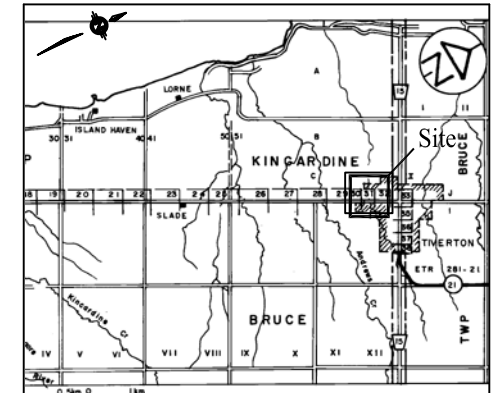
CONT No xxxx-xxxx
WP No GWP 408-94-00



Culvert # 2-476
Highway 21
BORE HOLE LOCATIONS & SOIL STRATA

SHEET
1

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

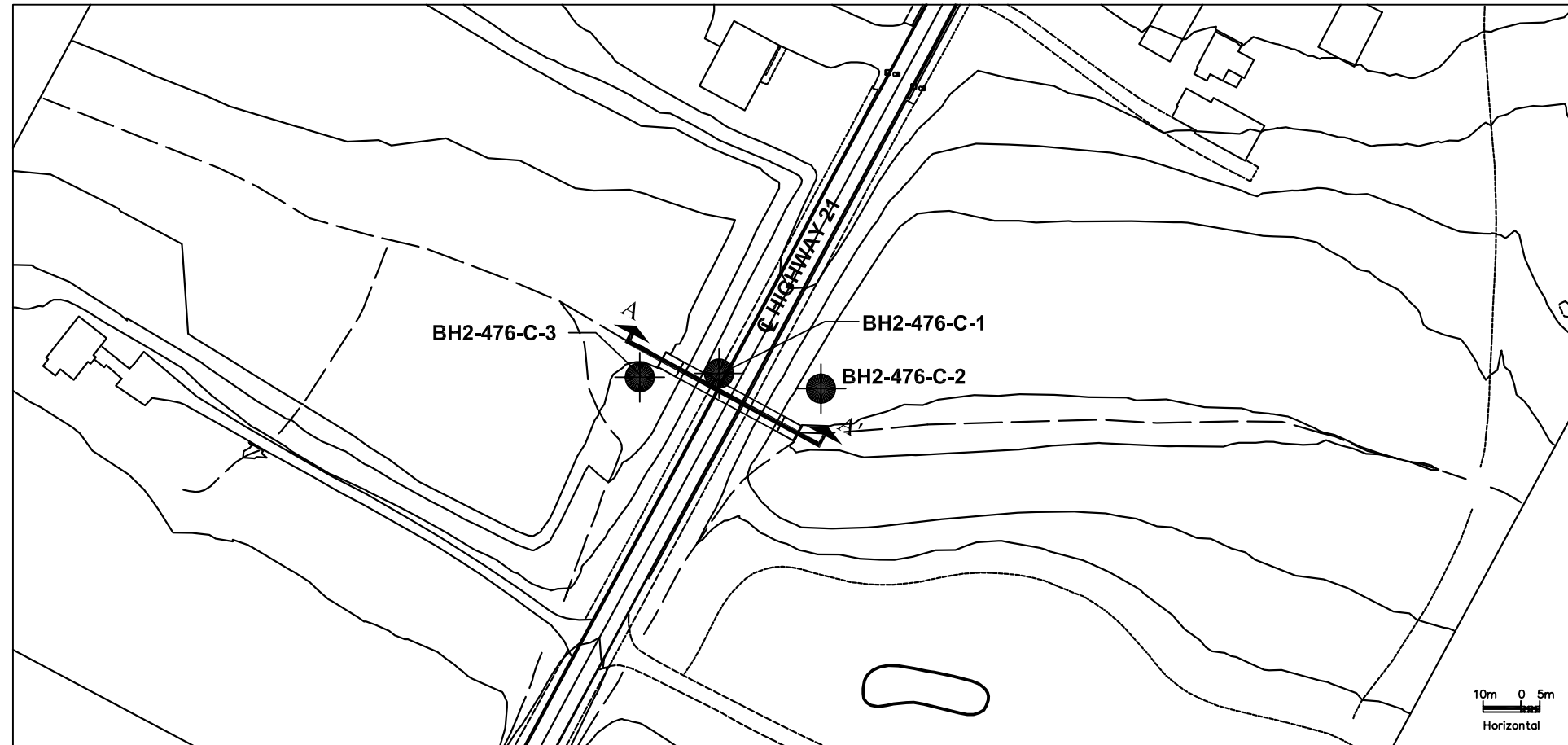


KEYPLAN

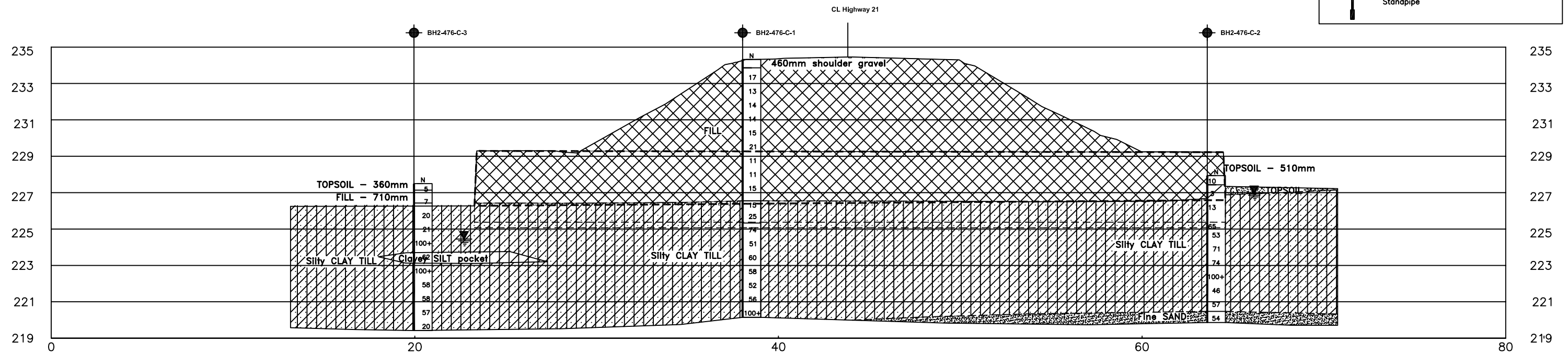
NTS

LEGEND

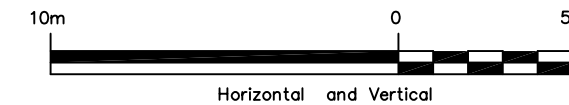
- Bore Hole
- Dynamic Cone Penetration Test (Cone)
- Bore Hole & Cone
- Blows/0.3m (Std Pen Test, 475 J/blow)
- Blows/0.3m (60° Cone, 475 J/blow)
- W L at time of investigation
- Standpipe



BOREHOLE LOCATION PLAN



SECTION A-A'
CENTERLINE OF CULVERT



REVISIONS	DATE	BY	DISCRIPTION
	DATE	BY	DISCRIPTION
	21/06/07	J.L.	Final
	15/01/07	J.L.	Draft

MTO GEOCRES No. 41A-191			
HWY No.	HWY 21		DIST Owen Sound
SUBM'D J.L.	CHECKED E.C.	DATE 21/06/07	SITE 2-476-C
DRAWN J.L.	CHECKED J.L.	APPROVED E.C.	DWG 1

- NOTES**
1. 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.
 2. THE BOUNDARIES BETWEEN SOIL STRATA HAVE BEEN ESTABLISHED ONLY AT BOREHOLE LOCATIONS. BETWEEN BOREHOLES AND BOUNDARIES ARE ASSUMED FROM GEOLOGICAL EVIDENCE.
 3. SUBGRADE ELEVATION OF THE EXISTING FOOTING NOT KNOWN AND IS ESTIMATED TO BE AT 1.2m BELOW THE CREEK BED.
 4. THIS DRAWING IS FOR SUBSURFACE INFORMATION ONLY. SURFACE DETAILS AND FEATURES ARE FOR CONCEPTUAL ILLUSTRATION.

BOREHOLE NO.	ELEVATION	UTM CO-ORDINATES	
		NORTH	EAST
2-476-C-1	234.32	4902864	380805
2-476-C-2	227.94	4902860	380832
2-476-C-3	227.50	4902863	380784

Ministry of Transportation/SNC-LAVALIN
G.W.P. 408-94-00
Reconstruction of Highway 21 from Kincardine northerly to Tiverton
Agreement # 3005-E-0038

06-8-IEG2-476
Final Report
Appendix A
September 28, 2007

Appendix A

Explanation of Terms Used in Report

Record of Borehole Sheet

Boreholes 2-476-C1 to 2-476-C3

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

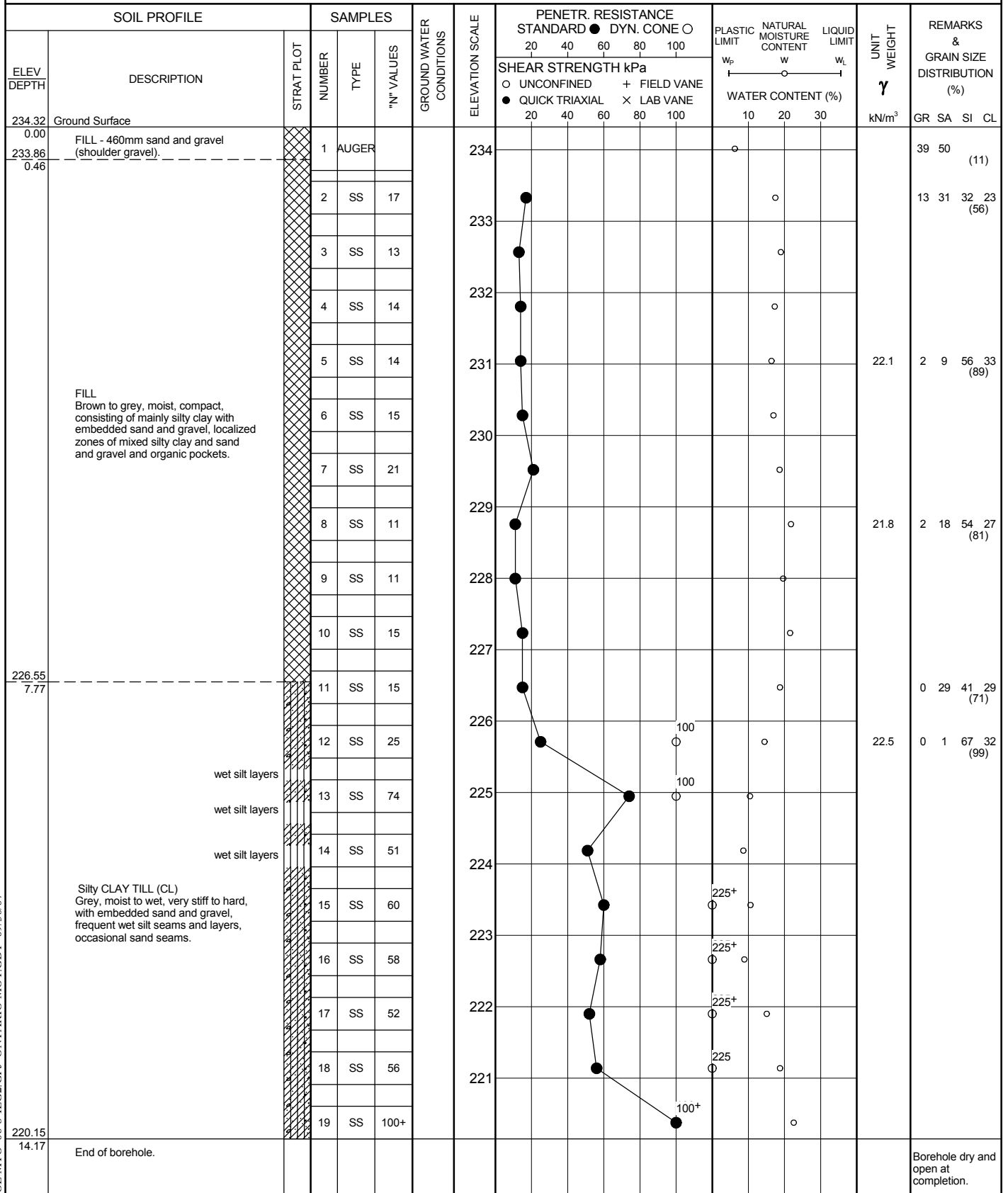
ρ_s	kg/m ³	DENSITY OF SOLID PARTICLES	e	1. %	VOID RATIO	e_{min}	1. %	VOID RATIO IN DENSEST STATE
γ_s	kn/m ³	UNIT WEIGHT OF SOLID PARTICLES	n	1. %	POROSITY	I_D	1	DENSITY INDEX = $\frac{e_{max} - e}{e_{max} - e_{min}}$
ρ_w	kg/m ³	DENSITY OF WATER	w	1. %	WATER CONTENT	D	mm	GRAIN DIAMETER
γ_w	kn/m ³	UNIT 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 2-476-C-1

1 OF 1

METRIC

W.P. GWP 408-94-00 LOCATION Site No. 2-476-C Northing - 4902864, Easting - 380805 ORIGINATED BY RB
 DIST Owen Sound HWY 21 BOREHOLE TYPE 100mm SST Auger COMPILED BY JL
 DATUM Geodetic DATE 09.25.06 - 09.25.06 CHECKED BY EC



JOE MTO 06-8-IEG2.GPJ ONTARIO MOT.GDT 09/30/07

+ 3, X 3: Numbers refer to Sensitivity

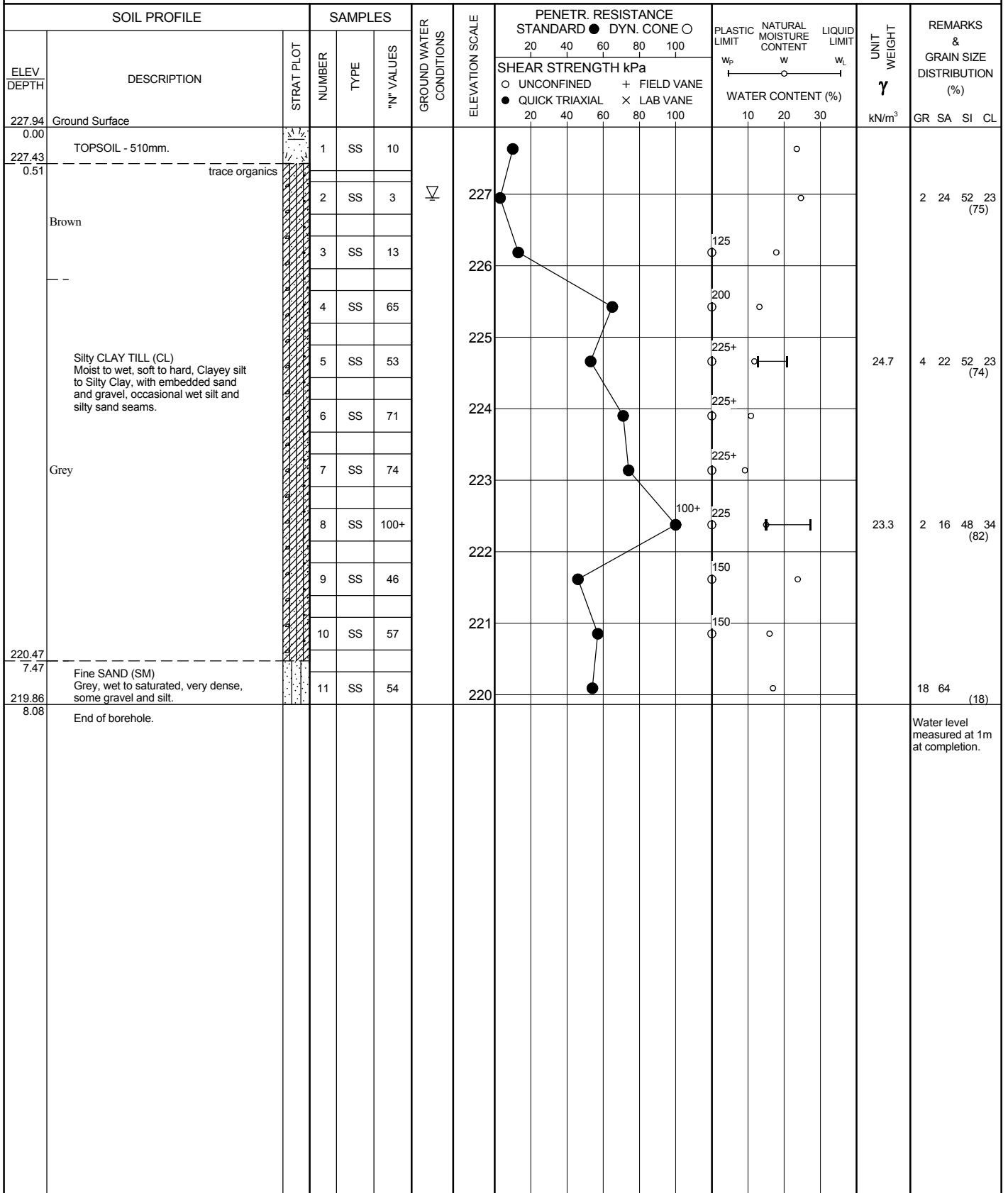
○ 150 UNCONFINED SHEAR STRENGTH INFERRED FROM POCKET PENETROMETER READINGS

RECORD OF BOREHOLE No 2-476-C-2

1 OF 1

METRIC

W.P. GWP 408-94-00 LOCATION Site No. 2-476-C Northing - 4902860, Easting - 380832 ORIGINATED BY RB
 DIST Owen Sound HWY 21 BOREHOLE TYPE 100mm SST Auger COMPILED BY JL
 DATUM Geodetic DATE 09.26.06 - 09.26.06 CHECKED BY EC



+ 3, X 3: Numbers refer to Sensitivity

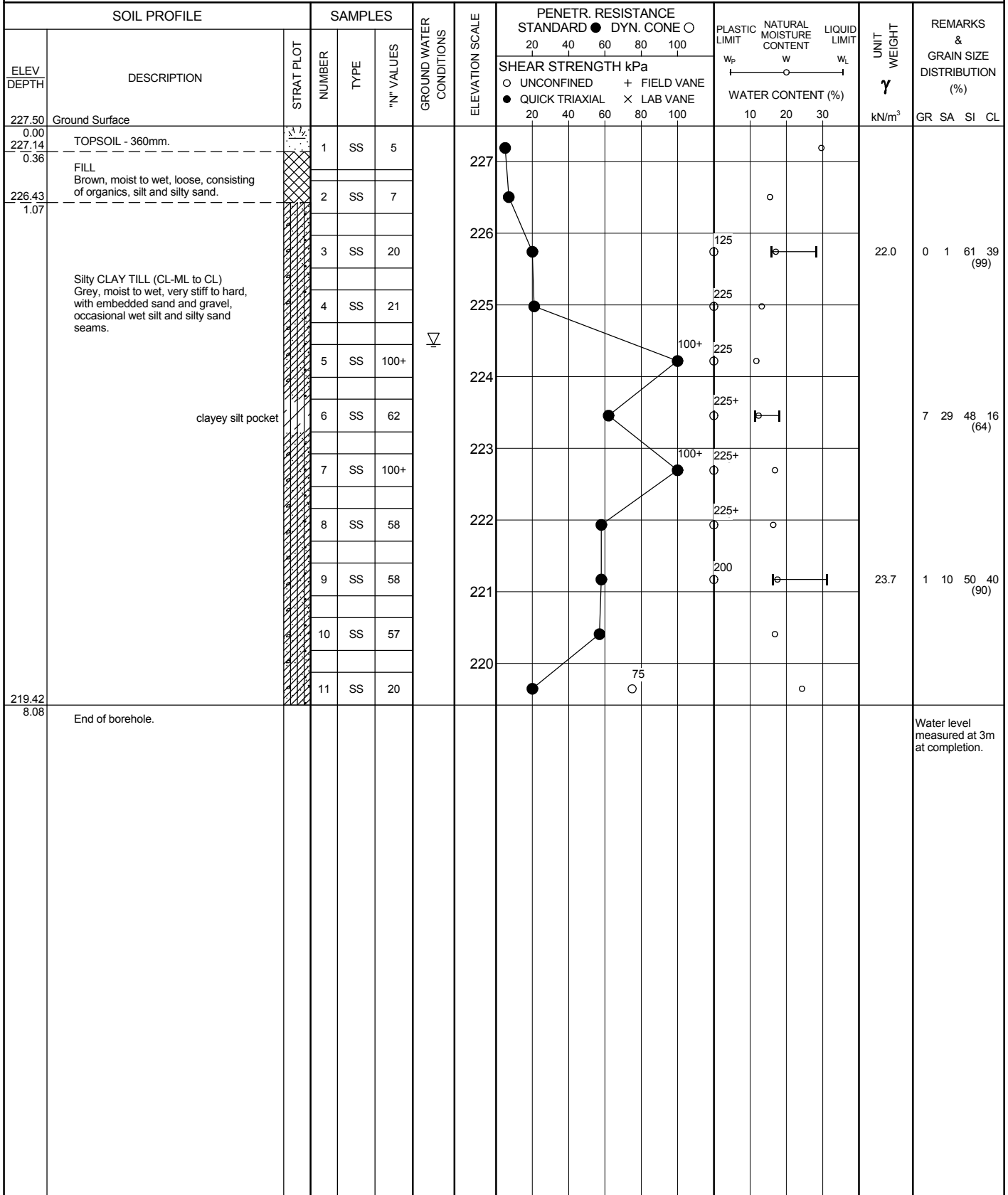
○ 150 UNCONFINED SHEAR STRENGTH INFERRED FROM POCKET PENETROMETER READINGS

RECORD OF BOREHOLE No 2-476-C-3

1 OF 1

METRIC

W.P. GWP 408-94-00 LOCATION Site No. 2-476-C Northing - 4902863, Easting - 380784 ORIGINATED BY RB
 DIST Owen Sound HWY 21 BOREHOLE TYPE 100mm SST Auger COMPILED BY JL
 DATUM Geodetic DATE 09.27.06 - 09.27.06 CHECKED BY EC



+ 3, X 3: Numbers refer to Sensitivity

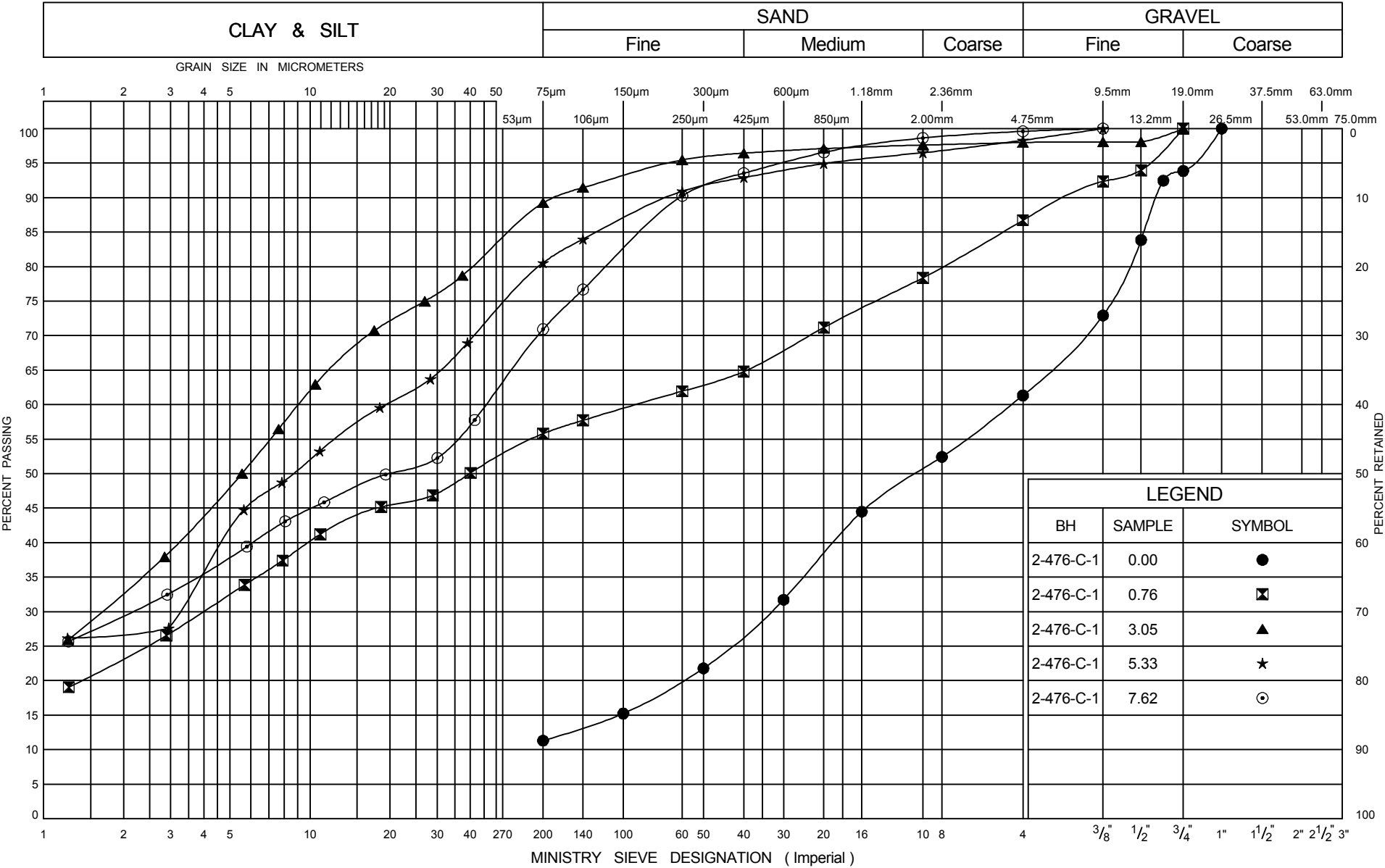
○ 150 UNCONFINED SHEAR STRENGTH INFERRED FROM POCKET PENETROMETER READINGS

Appendix B

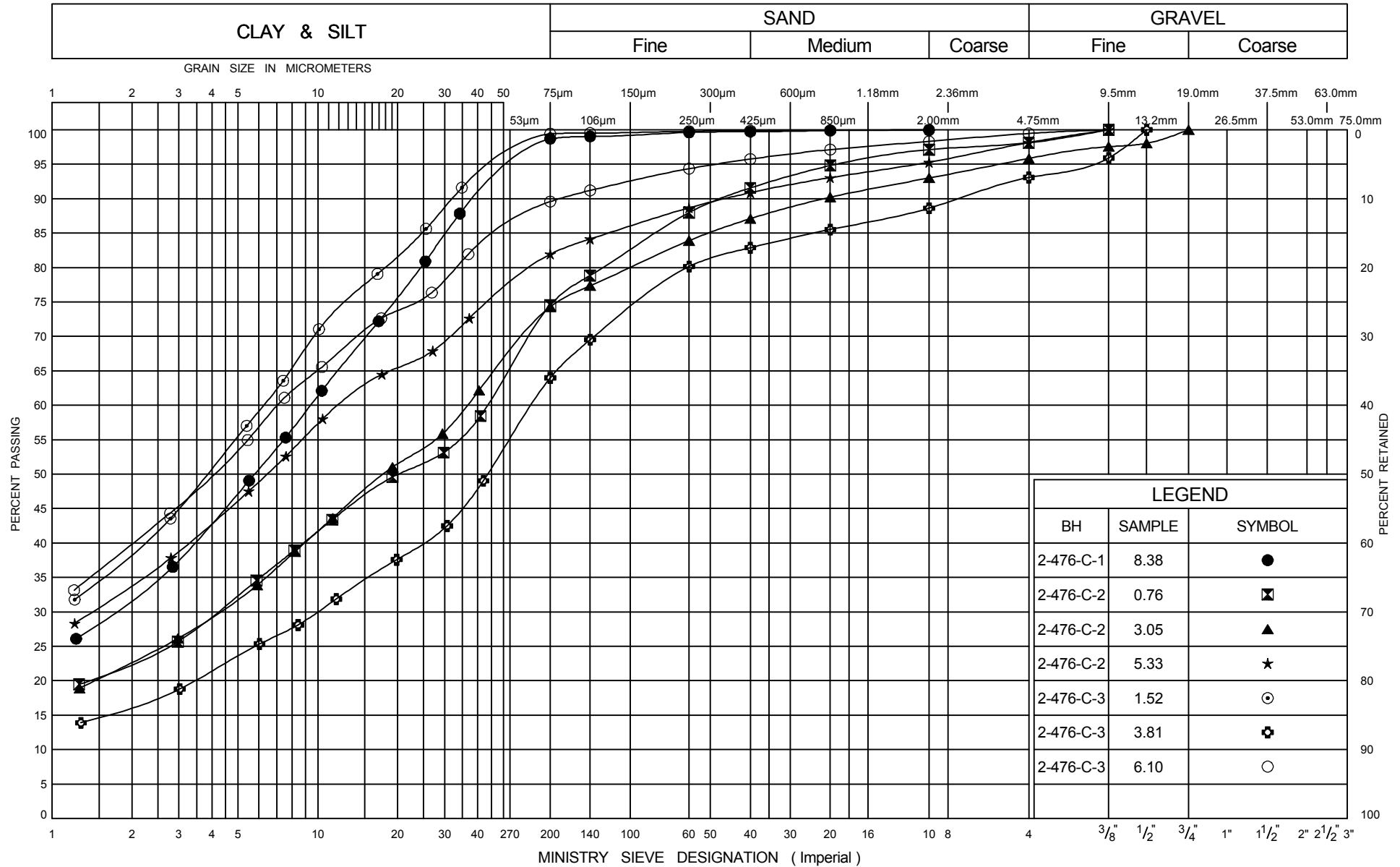
Laboratory Test Results

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

UNIFIED SOIL CLASSIFICATION SYSTEM



UNIFIED SOIL CLASSIFICATION SYSTEM



GRAIN SIZE DISTRIBUTION
SILTY CLAY TILL (CL)

FIG No 2

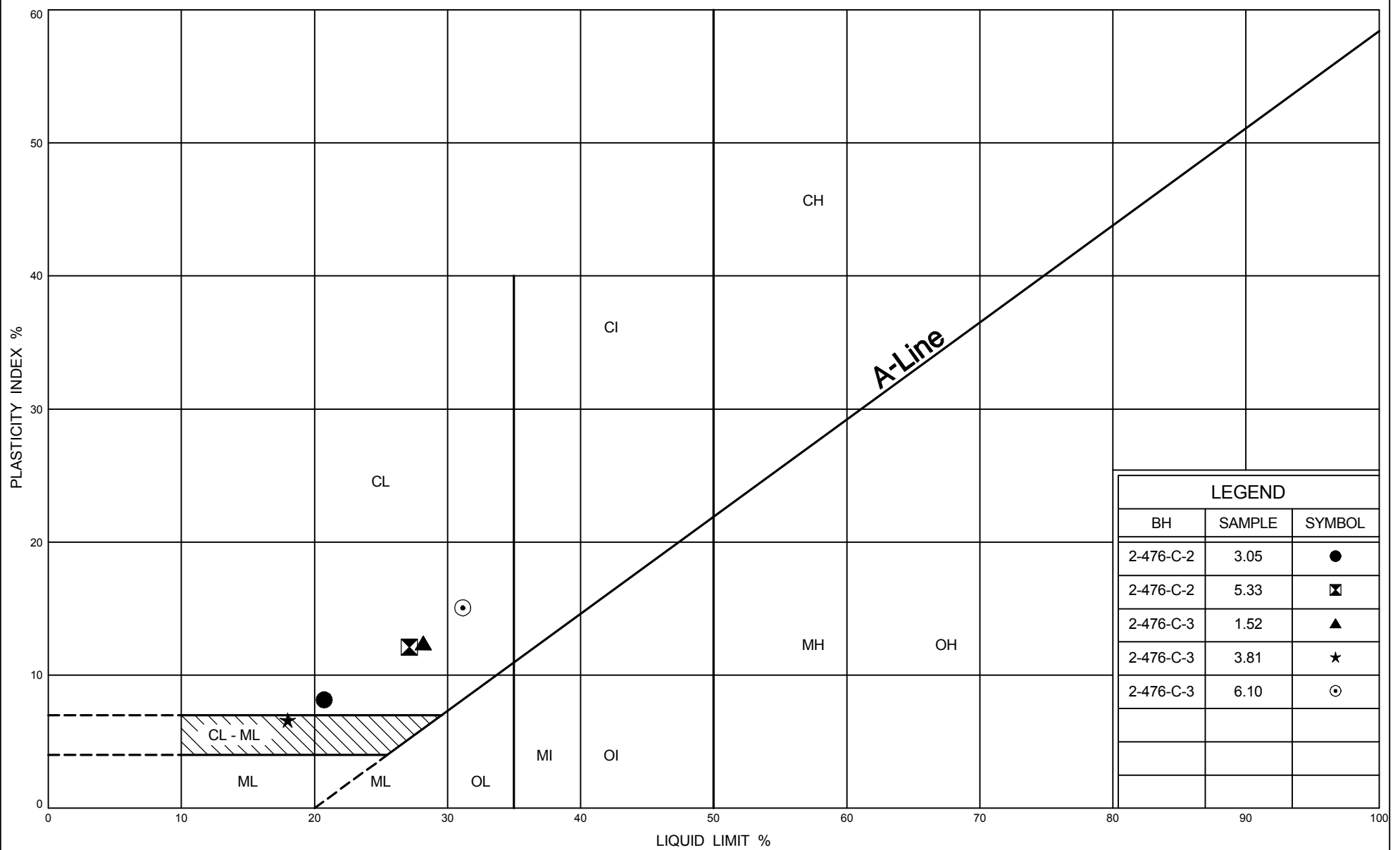
GWP 408-94-00

Highway 21-Kincardine to Tiverton



Ministry of
Transportation

Ontario



Ministry of
Transportation

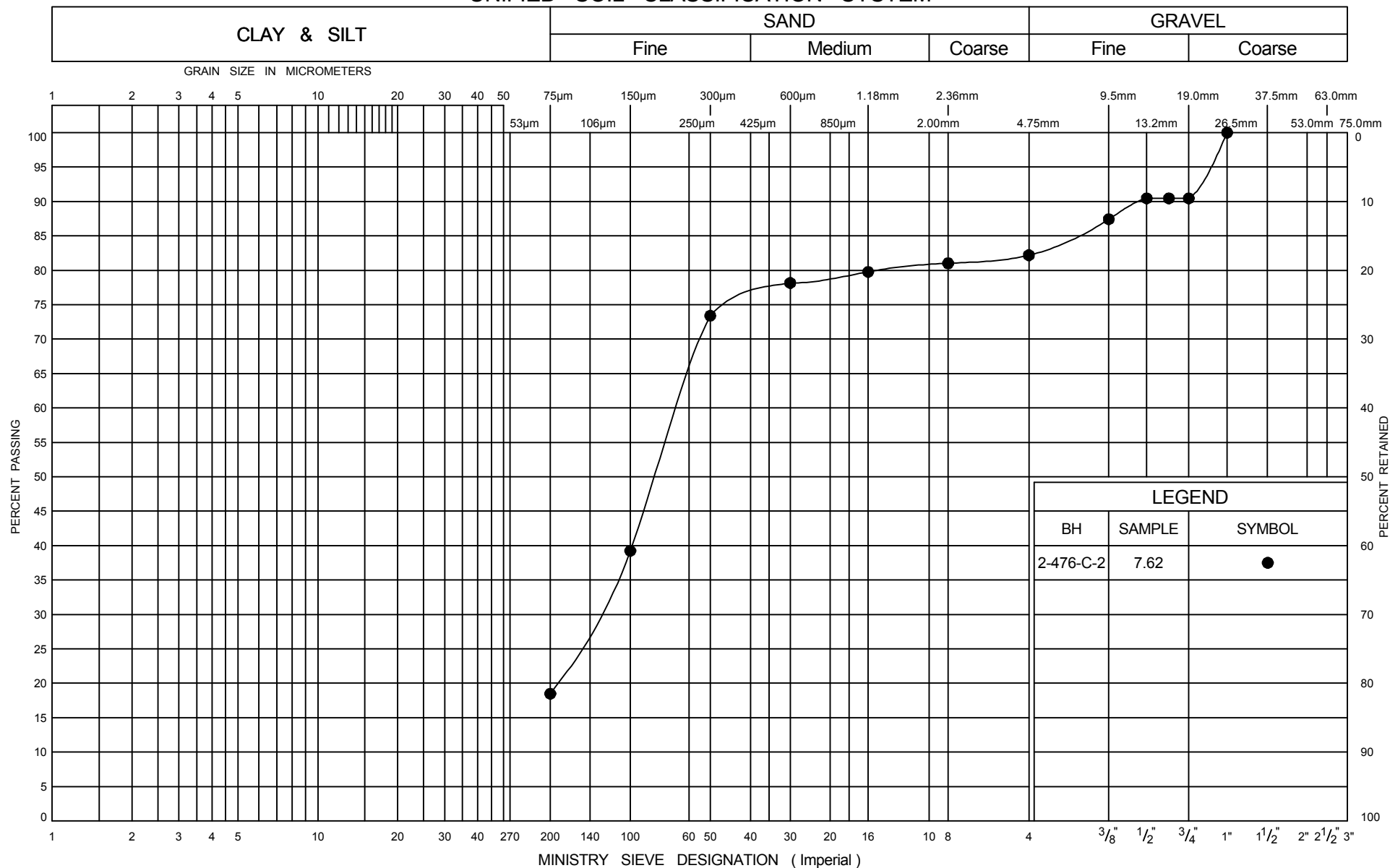
PLASTICITY CHART SILTY CLAY TILL (CL)

FIG No 3

GWP 408-94-00

Highway 21-Kincardine to Tiverton

UNIFIED SOIL CLASSIFICATION SYSTEM



Ministry of
Transportation

GRAIN SIZE DISTRIBUTION

Fine SAND, some silt (SP)

FIG No 4

GWP 408-94-00

Highway 21-Kincardine to Tiverton

Ministry of Transportation/SNC-LAVALIN
G.W.P. 408-94-00
Reconstruction of Highway 21 from Kincardine northerly to Tiverton
Agreement # 3005-E-0038

06-8-IEG2-476
Final Report
Appendix C
September 28, 2007

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/SNC-LAVALIN
G.W.P. 408-94-00
Reconstruction of Highway 21 from Kincardine northerly to Tiverton
Agreement # 3005-E-0038

06-8-IEG2-476
Final Report
Appendix D
September 28, 2007

Appendix D

Site Photographs



Culvert 2-476-C, general view



Culvert 2-476-C, inlet



Culvert 2-476-C, outlet