FOUNDATION INVESTIGATION
AND DESIGN REPORT
PROPOSED RECONSTRUCTION OF
STRUCTURAL CULVERT - SITE 2-473-C
HIGHWAY 21 FROM KINCARDINE TO TIVERTON
G.W.P. 408-94-00
Agreement # 3005-E-0038

I.E.

Geocres No.: 41A-189

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Prepared for:

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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 2-473-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 2-473-C is located on Highway 21, approximately 7.8 km north of Highway 9, located at station 21+366. 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 2.55 m high by 20.41 m long, with an overfill height of 2.1 m. The culvert opening dimensions were provided by AGM. Concrete headwalls exist at both ends of the culvert. Brown silt to clayey silt 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 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.

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 227.5 m while the ground surface at the base of the embankment and in the flood plain is between elevations 222.0 m and 222.5m.

3.0 INVESTIGATION PROCEDURES

3.1 Field Investigation

On September 13 and 14, 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 2-473-C-1 to 2-473-C-3 and the depths of sampling were as follows:

Borehole No.	Depth of Sampling (m)
2-473-C-1	8.08
2-473-C-2	8.08
2-473-C-3	10.36

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 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 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 sheets (Appendix "A"), and Laboratory Test Results (Figures 1 to 5, 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 dense to very dense sand, silt to silt till and/or stiff to hard clayey silt till.

4.1.1 Pavement, Fill, Topsoil

Borehole 2-473-C-3, which was located at the east edge of existing pavement in the shoulder area, encountered 510 mm shoulder gravel. Underlying the shoulder gravel is the embankment fill material that extended to a depth of 5.18 m (elevation 222.14 m). The fill beneath the shoulder gravel consists of a mixture of gravel, sand, silt and clay with organic inclusions and occasional sand and gravel pockets.

At Boreholes 2-473-C-1 and 2-473-C-2, topsoil fill and silty clay fill were contacted to depths of 0.76 m (elevation 222.69 m) and 0.53 m (elevation 222.54 m) respectively.

Two (2) typical grain size distributions of the embankment fill are shown on Figure 1 of Appendix “B”.

Standard penetration tests yielded “N”-values from 5 to 10 blows per 0.3 m. This fill is brown to grey in colour and the measured natural moisture contents range from 6 to 34%. Based on the above field and laboratory test results, together and tactile examination, the fill materials exhibited loose to compact compactness condition.

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

4.1.2 Sand, Silt, Silt Till

Sand, sandy silt to silty sand, silt and silt till, were encountered below the embankment fill at Borehole 2-473-C-3 and the topsoil and silty clay fill layers at Borehole 2-473-C-1. These grey silt/sand and silt till strata extended to the full depth of the boreholes (i.e., 8.08 m below the ground surface at Borehole 2-473-C-1, elevation 215.38 m).

A 2.89 m thick silt to sand and silt layer was sandwiched within the clayey silt and clayey silt fill at Borehole 2-473-C-2, beginning at a depth of 2.90 m. Clayey silt layers were present within the silt/sand and the silt till. Five (5) grain size analyses were performed and the results are plotted on Figure 2 of Appendix “B”.

Standard penetration tests yielded “N”-values from 52 to over 100 blows per 0.3 m. A localized low “N”-value of 9 blows per 0.3 m was recorded immediately below the fill at Borehole 2-473-C-1. The unit weight was measured to be between 22.9 and 23.5 kN/m^3. The natural moisture contents were in the range of 11 to 20%.

Two (2) samples were tested and exhibited the following Atterberg Limits. These results are shown in Figure 3 of Appendix “B” and summarized below:

Liquid Limit (W_L)	14%
Plastic Limit (W_P)	13 to 14%
Plasticity Index (I_p)	0 to 1%

Based on the above field and laboratory test results, together with visual and tactile examination, these fine granular deposits are considered to have dense to very dense compactness condition.

4.1.3 Clayey Silt to Clayey Silt Till

A stratum of grey clayey silt to clayey silt till was contacted below the topsoil fill and 125 mm thick sand layer at Borehole 2-473-C-2, and extended to the full depth of the borehole (i.e., 8.08 below the ground surface at Borehole 2-473-C-2 m, elevation 214.99 m). A 2.89 m thick silt to sand and silt layer was sandwiched within the clayey silt, beginning at a depth of 2.90 m. Three (3) grain size analyses were performed on the clayey silt layers and the results are presented on Figure 4 of Appendix "B".

Standard penetration tests yielded "N"-values from 11 to 88 blows per 0.3 m. Undrained shear strength as determined from field pocket penetrometer yielded results of over 225 kPa. A single unit weight was measured on the clayey silt and yielded a result of 23.5 kN/m^3.

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)	17 to 19%, average at 17.6%
Plastic Limit (W_P)	12 to 14%, average at 12.7%
Plasticity Index (I_p)	5%

The natural moisture contents were in the range of 12 to 22%. These results are characteristic of clayey soils of low plasticity (CL-ML). 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 clayey silt to clayey silt till deposit exhibited generally very stiff to hard consistency, with a stiff consistency within the upper 0.6 m of this stratum Borehole 2-473-C-2.

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 2-473-C-1, 2-473-C-2 and 2-473-C-3 at depths of 1.2, 0.6 and 5.0 m respectively below ground surface, corresponding to elevations 222.25, 222.47 and 222.32 m. The water entered the boreholes from native saturated sand to silty sand layers.

The water level in the creek was approximately 0.3m 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 to silt soils, and on top of the clayey to sandy silt 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 2-473-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 2-473-C is located on Highway 21, approximately 7.8 km north of Highway 9, located at station 21+366. The existing structure is a reinforced concrete, rigid frame open footing culvert having dimensions of 3.60 m wide by 2.55 m high by 20.41 m long, with an overfill height of 2.1 m. The culvert opening dimensions were provided by AGM. Concrete headwalls exist at both ends of the culvert. Brown silt to clayey silt 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 north and south sides of the culvert, with a maximum height of approximately 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 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 clayey silt and dense to very dense silty sand to sandy silt.

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 silt/sand or clayey silt at the elevation and bearing resistances shown below:

Highest Elevation (m)	**Factored Geotechnical Resistance at ULS (kPa)**	**Geotechnical Reaction at SLS (kPa)**
221.90	800	400

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.

The groundwater level was near the ground surface at the time of the investigation. 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 1.2 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. The silt to sand and silt subgrade could easily be disturbed under wet conditions. 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 precast concrete culvert. The Granular “A” layer should be compacted to 98% of the material’s standard Proctor maximum dry density (SPMDD). The sand to silt subgrade could easily be disturbed under wet conditions. Any disturbed subgrade material should be removed and replaced with additional compacted bedding material. 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 silt/sand or clayey silt at the elevation and bearing resistances shown below:

Highest Elevation (m)	Factored Geotechnical Resistance at ULS (kPa)	Geotechnical Reaction at SLS (kPa)
221.90	800	400

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 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 (γ h + q)

Where:

K = earth pressure coefficient, use value from table below

γ = unit weight of soil, = 21.2 kN/m^3 for Granular “B”
= 22.8 kN/m^3 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” $\phi = 35^o$	**Granular “B”** ϕ **= 30 to 35**^{o}
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 clayey silt 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 clayey silt as per Table 24.4 CFEM 4th Edition, 2006.

As silty sand to sandy silt will also form part of the founding subgrade, the unfactored horizontal resistance can be calculated using an angle of friction of 24 degrees or a friction factor of 0.45. The lower factored horizontal resistance should be used in the design. 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 5 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.

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 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 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 21 is between 5 and 6.5 m.

Excavation to depths of up to 6.5 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. Saturated sand (Borehole 2-473-C-1) and silty sand to sandy silt (Borehole 2-473-C-3) will be encountered during excavation, and groundwater control will be required to handle surface runoff and minor seepage. The 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 loose to compact sandy soils encountered at this site are classified as Type 3 soils and the very stiff to hard clayey silt 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 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.

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. Fine sand, silt and clayey silt could be exposed at the streambed, and their permissible velocities are 0.6 m/s, 0.8 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).

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

Drawing 1

Borehole Locations

And

Soil Strata

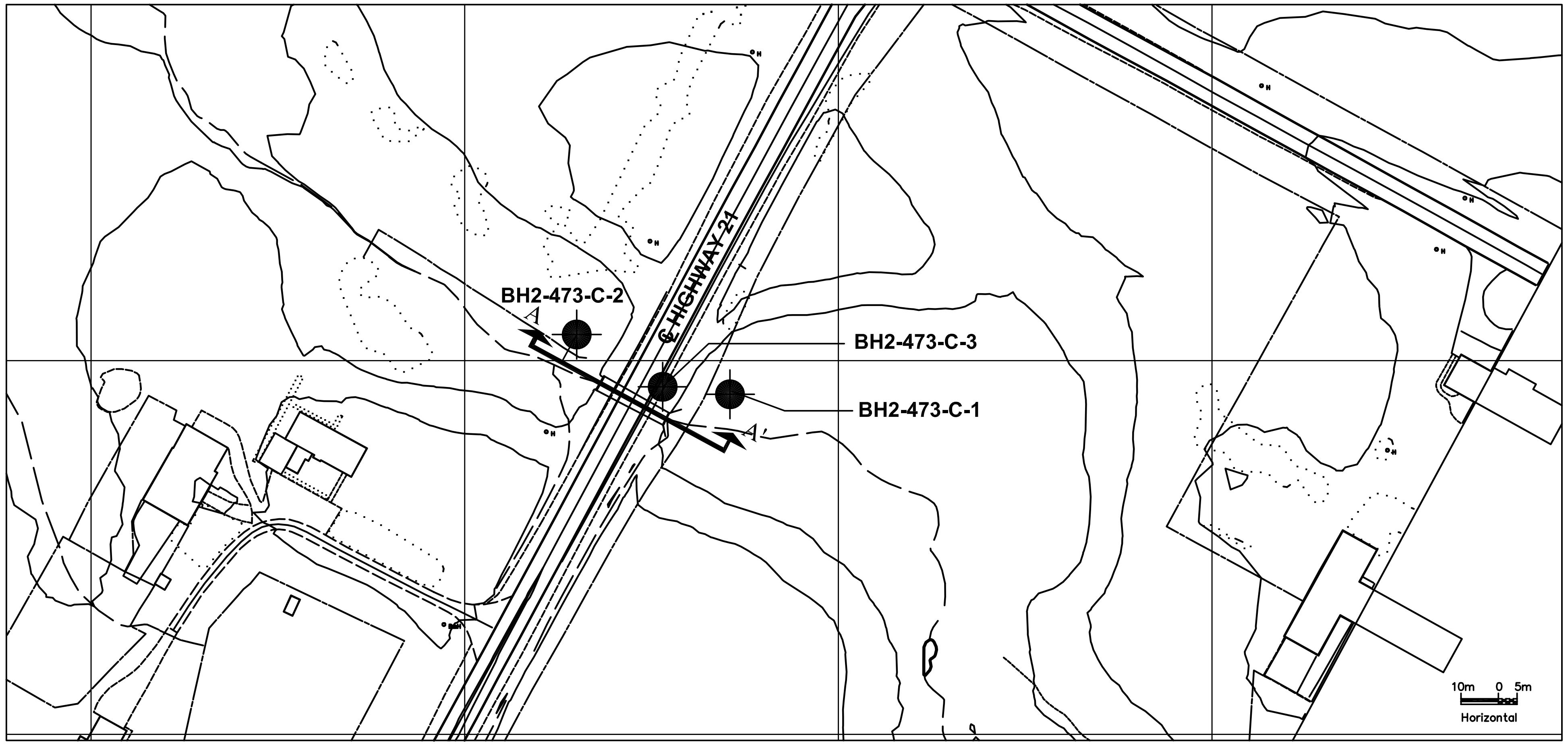
METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN

CONT No xxxx–xxxx
WP No GWP 408–94–00

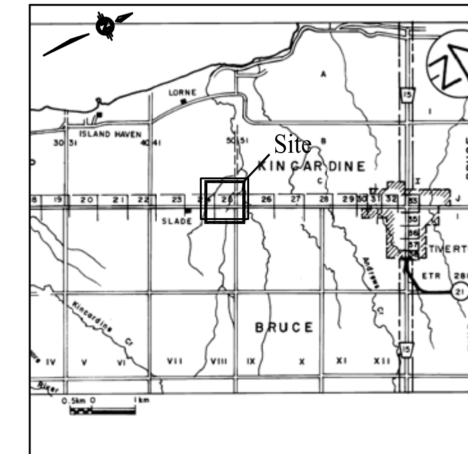
Culvert # 2–473
Highway 21
BORE HOLE LOCATIONS & SOIL STRATA

SHEET 1

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



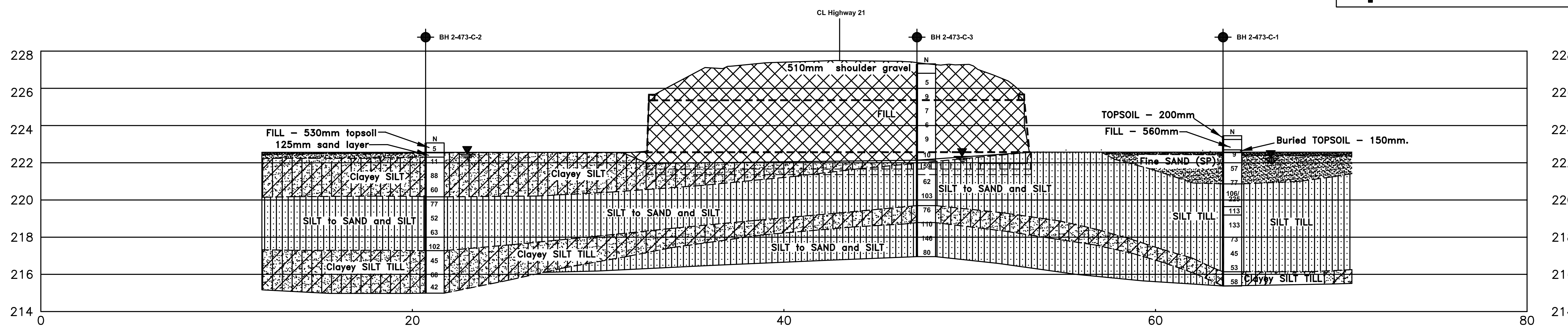
BOREHOLE LOCATION PLAN



KEYPLAN

LEGEND

	Bore Hole	NTS
	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	



SECTION A–A'
CENTERLINE OF CULVERT

10m 0 5m
Horizontal and Vertical

REVISIONS	DATE	BY	DISCRIPTION
	28/09/07	J.L.	Final
	17/01/07	J.L.	Draft

MTO GEOCRES No. 41A–189

HWY No.	HWY 21		DIST	Owen Sound
SUBM'D J.L.	CHECKED E.C.	DATE 21/06/07	SITE	2–473–C
DRAWN J.L.	CHECKED J.L.	APPROVED E.C.	DWG	1

BOREHOLE NO.	ELEVATION	UTM CO–ORDINATES NORTH	EAST
2–473–C–1	223.45	4899791	379171
2–473–C–2	223.07	4899807	379130
2–473–C–3	227.32	4899793	379153

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.

Appendix A

Explanation of Terms Used in Report

Record of Borehole Sheet

Boreholes 2-473-C1 to 2-473-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 'A' SIZE DRILL RODS. THE RESISTANCE TO CONE PENETRATION IS MEASURED AS THE NUMBER OF BLOWS FOR EACH 0.3m ADVANCE OF THE CONICAL POINT INTO THE UNDISTURBED GROUND.

SOILS ARE DESCRIBED BY THEIR COMPOSITION AND CONSISTENCY OR DENSENESS.

CONSISTENCY: COHESIVE SOILS ARE DESCRIBED ON THE BASIS OF THEIR UNDRAINED SHEAR STRENGTH (c_u) AS FOLLOWS:

c_u (kPa)	0 - 12	12 - 25	25 - 50	50 - 100	100 - 200	>200
	VERY SOFT	*SOFT*	*FIRM*	*STIFF*	*VERY STIFF*	*HARD*

DENSENESS: COHESIONLESS SOILS ARE DESCRIBED ON THE BASIS OF DENSENESS AS INDICATED BY SPT N VALUES AS FOLLOWS:

N (BLOWS/0.3m)	0 - 5	5 - 10	10 - 30	30 - 50	>50
	VERY LOOSE	*LOOSE*	*COMPACT*	*DENSE*	*VERY DENSE*

ROCKS ARE DESCRIBED BY THEIR COMPOSITION AND STRUCTURAL FEATURES AND / OR STRENGTH.

RECOVERY: SUM OF ALL RECOVERED ROCK CORE PIECES FROM A CORING RUN EXPRESSED AS A PERCENT OF THE TOTAL LENGTH OF THE CORING RUN.

MODIFIED RECOVERY: SUM OF THOSE INTACT CORE PIECES, 100mm+ IN LENGTH EXPRESSED AS A PERCENT OF THE LENGTH OF THE CORING RUN. THE ROCK QUALITY DESIGNATION (R Q D), FOR MODIFIED RECOVERY, IS:

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

e	1, %	VOID RATIO
n	1, %	POROSITY
w	1, %	WATER CONTENT
S_r	%	DEGREE OF SATURATION
w_L	%	LIQUID LIMIT
w_P	%	PLASTIC LIMIT
w_s	%	SHRINKAGE LIMIT
I_P	%	PLASTICITY INDEX $= w_L - w_P$
I_L	1	LIQUIDITY INDEX $= \frac{w - w_P}{I_P}$
I_C		CONSISTENCY INDEX $= \frac{w_L - w}{I_P}$
e_{max}	1, %	VOID RATIO IN LOOSEST STATE

e_{min}	1, %	VOID RATIO IN DENSEST STATE
I_D	1	DENSITY INDEX $= \frac{e_{max} - e}{e_{max} - e_{min}}$
D	mm	GRAIN DIAMETER
D_n	mm	n PERCENT - DIAMETER
C_U	1	UNIFORMITY COEFFICIENT
h	m	HYDRAULIC HEAD OR POTENTIAL
q	m^3/s	RATE OF DISCHARGE
v	m/s	DISCHARGE VELOCITY
i	1	HYDRAULIC GRADIENT
k	m/s	HYDRAULIC CONDUCTIVITY
j	kN/m^3	SEEPAGE FORCE

RECORD OF BOREHOLE No 2-473-C-1

1 OF 1 **METRIC**

W.P. GWP 408-94-00 LOCATION Site No. 2-473-C Northing - 4899791, Easting - 379171 ORIGINATED BY RB

DIST Owen Sound HWY 21 BOREHOLE TYPE 100mm SST Auger COMPILED BY JL

DATUM Geodetic DATE 09.13.06 - 09.13.06 CHECKED BY EC

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	WATER CONTENT (%)	UNIT WEIGHT γ (kN/m³)	GR	SA	SI	CL
223.45	Ground Surface									
0.00	TOPSOIL - 200mm. FILL Brown, moist, consisting mainly of silty clay.	1	AUGER							
222.69 / 0.76	Buried TOPSOIL - 150mm. Fine SAND (SP) Grey, wet to saturated, loose to very dense, some silt.	2	SS	9						
		3	SS	57			0	79	18	3 (21)
		4	SS	77						
220.86 / 2.59	SILT TILL (ML) Grey, moist, dense to very dense, with embedded sand and gravel, interbedded with sand, silty sand and clayey silt layers.	5	SS	106/225		22.9	0	14	75	10 (86)
	very dense sand layer	6	SS	113						
		7	SS	133						
		8	SS	73			0	13	70	16 (86)
		9	SS	45						
		10	SS	53						
	clayey silt layer	11	SS	58		23.5	0	6	70	24 (94)
215.37 / 8.08	End of Borehole.									

Ground water conditions: water level symbol at approx. elev. 222.3.

Penetration resistance plot annotations: 100+ (samples 5, 6, 7); 225+ (unconfined, sample 11).

REMARKS: Water level measured at 1.2m at completion.

+3, ×3: Numbers refer to Sensitivity

○150 UNCONFINE SHEAR STRENGTH INFERRED FROM POCKET PENETROMETER READINGS

Ministry of Transportation
Ontario

Foundation Design

RECORD OF BOREHOLE No 2-473-C-2

METRIC

W.P. GWP 408-94-00 LOCATION Site No. 2-473-C Northing - 4899807, Easting - 379130 ORIGINATED BY RB

DIST Owen Sound HWY 21 BOREHOLE TYPE 100mm SST Auger COMPILED BY JL

DATUM Geodetic DATE 09.13.06 - 09.13.06 CHECKED BY EC

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
223.07	Ground Surface						
0.00	FILL - 530mm, consisting mainly of topsoil.	1	SS	5	∇		
222.54 / 0.53	125mm sand layer						
	Clayey SILT (CL-ML) Grey, moist to wet, stiff to hard, with embedded sand and gravel.	2	SS	11			
		3	SS	88			4 16 56 24 (80)
		4	SS	60			
220.17 / 2.90	SILT to Sand and SILT (ML to SM) Grey, wet, very dense.	5	SS	77			
		6	SS	52			2 52 43 3 (45)
		7	SS	63			
		8	SS	102			
217.28 / 5.79	Clayey SILT TILL (CL-ML) Grey, moist, hard, with embedded sand and gravel.	9	SS	45		23.5	0 3 85 12 (97)
		10	SS	68			
		11	SS	42			
214.99 / 8.08	End of Borehole.						Water level measured at 0.6m at completion.

Elevation scale: 222, 221, 220, 219, 218, 217, 216, 215

PENETR. RESISTANCE: STANDARD ● DYN. CONE ○ (20, 40, 60, 80, 100)
SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE (20, 40, 60, 80, 100)
Annotations: 100+ (sample 8); 225+ (samples 9 and 11)

PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L
WATER CONTENT (%) 10 20 30

+ 3, × 3: Numbers refer to Sensitivity

○ 150 UNCONFINE SHEAR STRENGTH INFERRED FROM POCKET PENETROMETER READINGS

RECORD OF BOREHOLE No 2-473-C-3

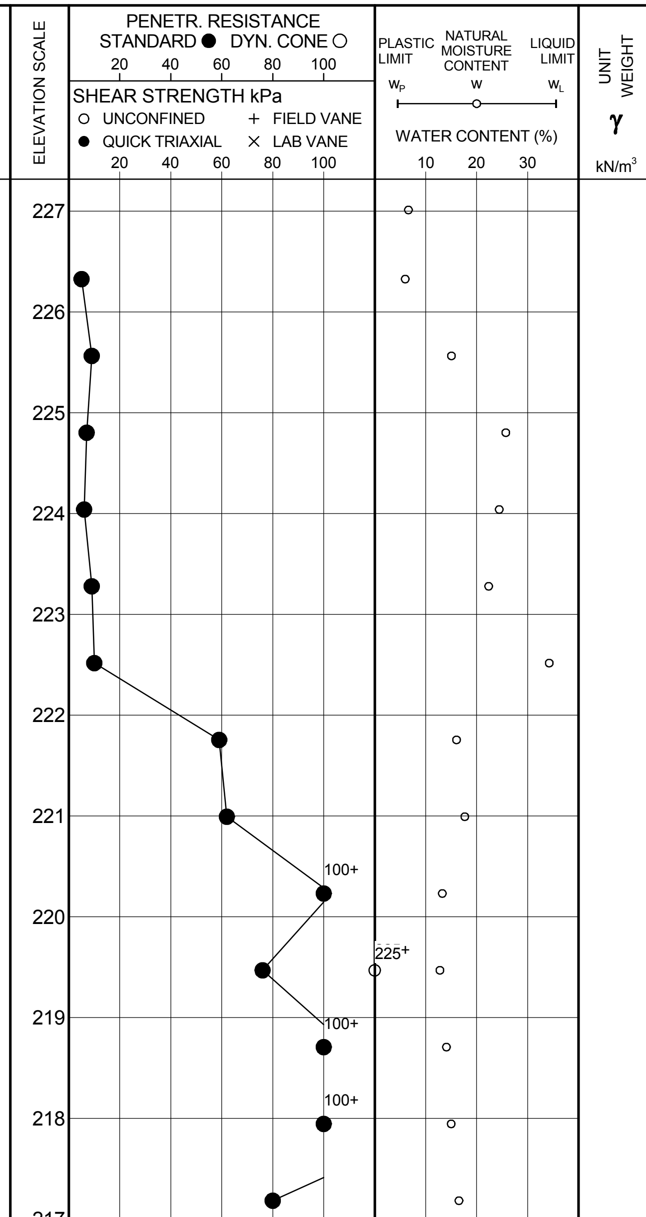
METRIC

W.P. GWP 408-94-00 LOCATION Site No. 2-473-C Northing - 4899793, Easting - 379153 ORIGINATED BY RB

DIST Owen Sound HWY 21 BOREHOLE TYPE 100mm SST Auger COMPILED BY JL

DATUM Geodetic DATE 09.14.06 - 09.14.06 CHECKED BY EC

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
227.32	Ground Surface					
0.00	FILL - 510mm, sand and gravel (shoulder gravel)	1	AUGER			
226.81 / 0.51	FILL Brown to grey, moist, loose to compact, consisting of gravel, sand silt and clay, with organic inclusions and occasional sand and gravel pockets.	2	SS	5		66 27 (7)
		3	SS	9		
		4	SS	7		
		5	SS	6		13 25 40 22 (62)
		6	SS	9		
		7	SS	10	▽	
222.14 / 5.18	Silty SAND to Sandy SILT (SM to ML) Grey, wet, very dense, interbedded, occasional silty clay seams and layers.	8	SS	59		
		9	SS	62		
		10	SS	103		
	silty clay layer	11	SS	76		
		12	SS	110		0 51 45 4 (49)
		13	SS	146		
		14	SS	80		
216.96 / 10.36	End of Borehole.					Water level measured at 5m at completion.



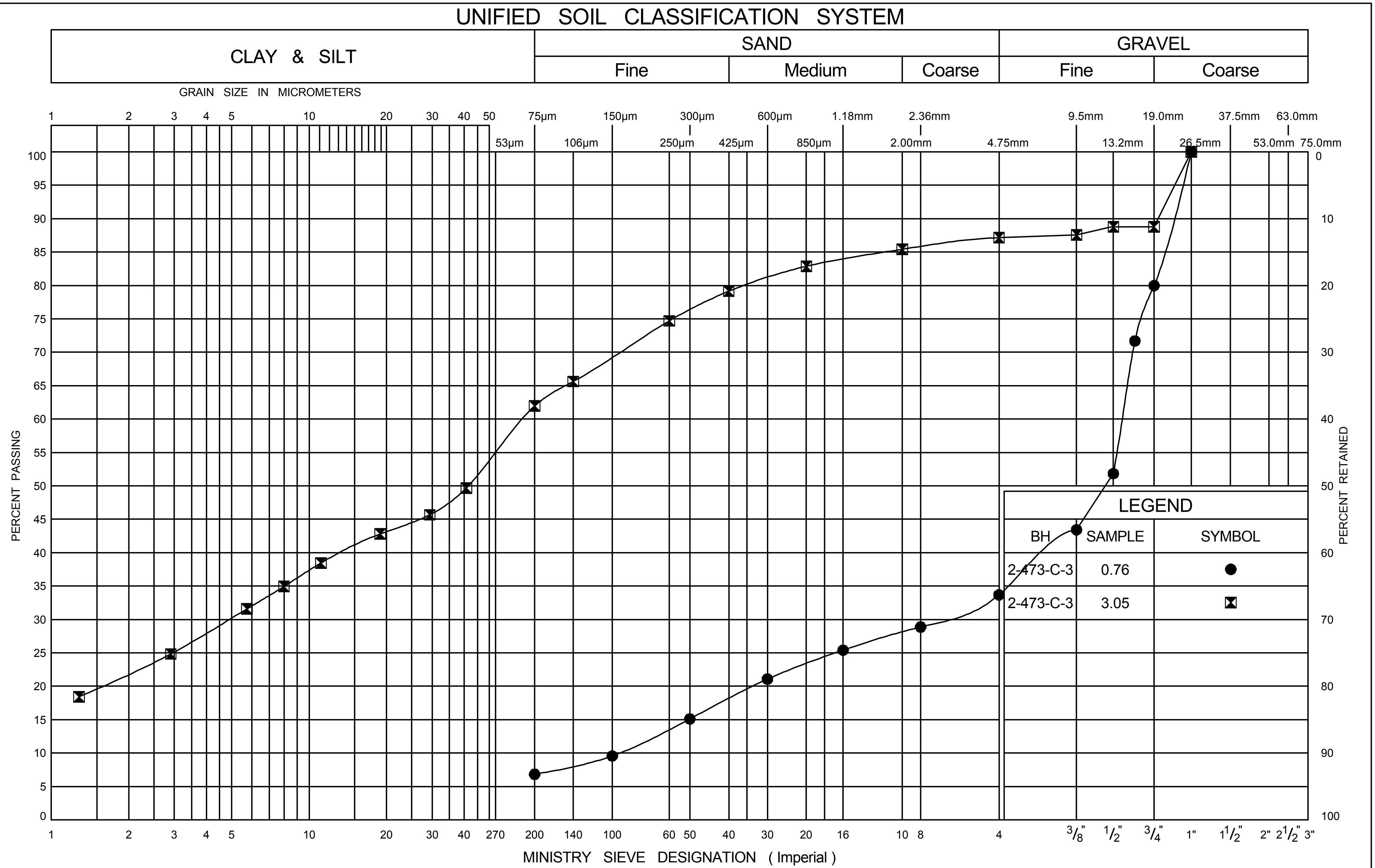
+ ^{3}, × ^{3}: Numbers refer to Sensitivity ○ ^{150} UNCONFINE SHEAR STRENGTH INFERRED FROM POCKET PENETROMETER READINGS

Appendix B

Laboratory Test Results

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

78 12 M



ONTARIO MOT GRAIN SIZE LARGE CULVERTS 06-8-IEG2.GPJ ONTARIO MOT.GDT 6/22/07

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GRAIN SIZE DISTRIBUTION

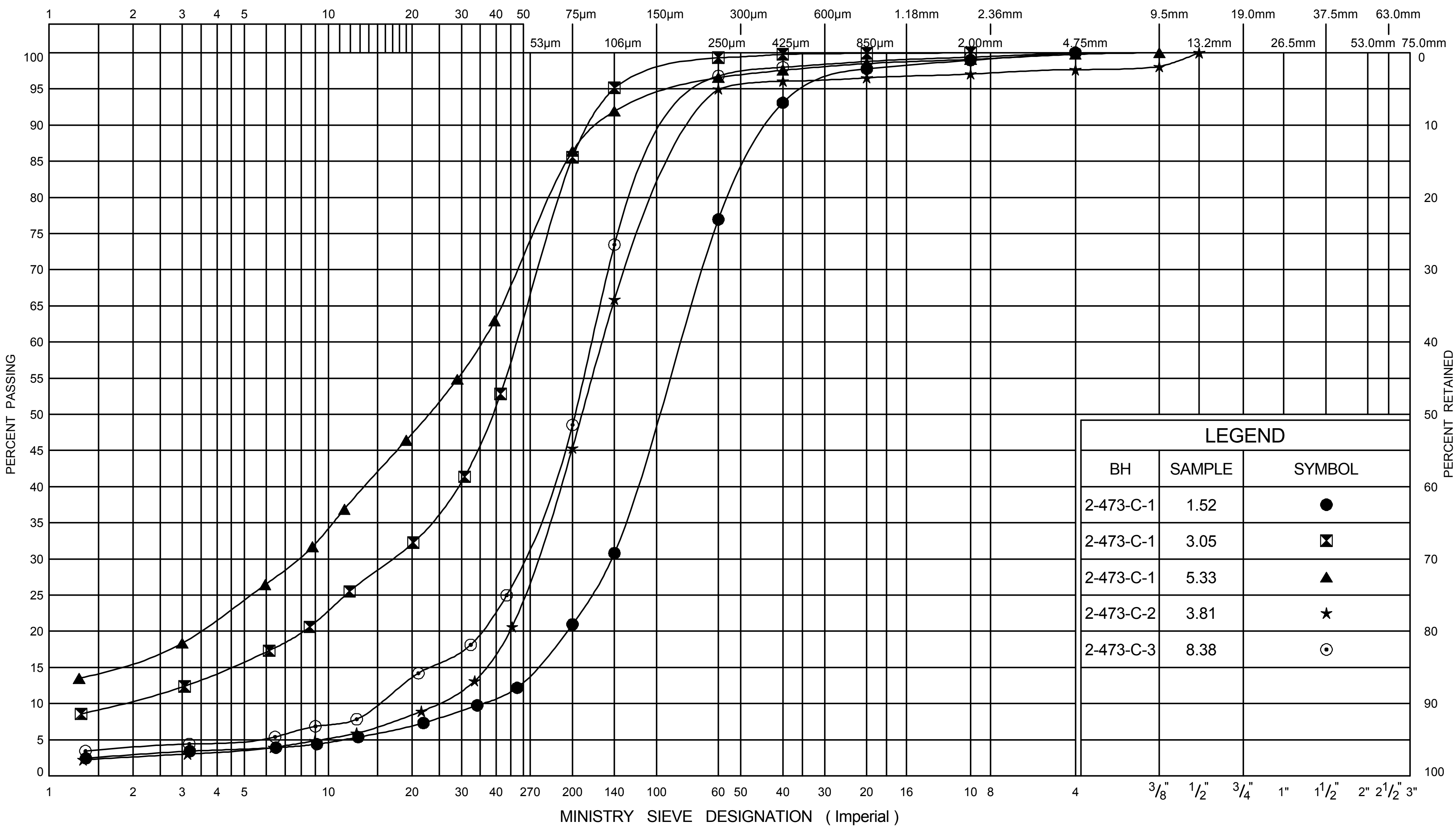
FILL

FIG No 1

GWP 408-94-00

Highway 21-Kincardine to Tiverton

78 12 M



LEGEND

BH	SAMPLE	SYMBOL
2-473-C-1	1.52	●
2-473-C-1	3.05	⊠
2-473-C-1	5.33	▲
2-473-C-2	3.81	★
2-473-C-3	8.38	⊙

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GRAIN SIZE DISTRIBUTION

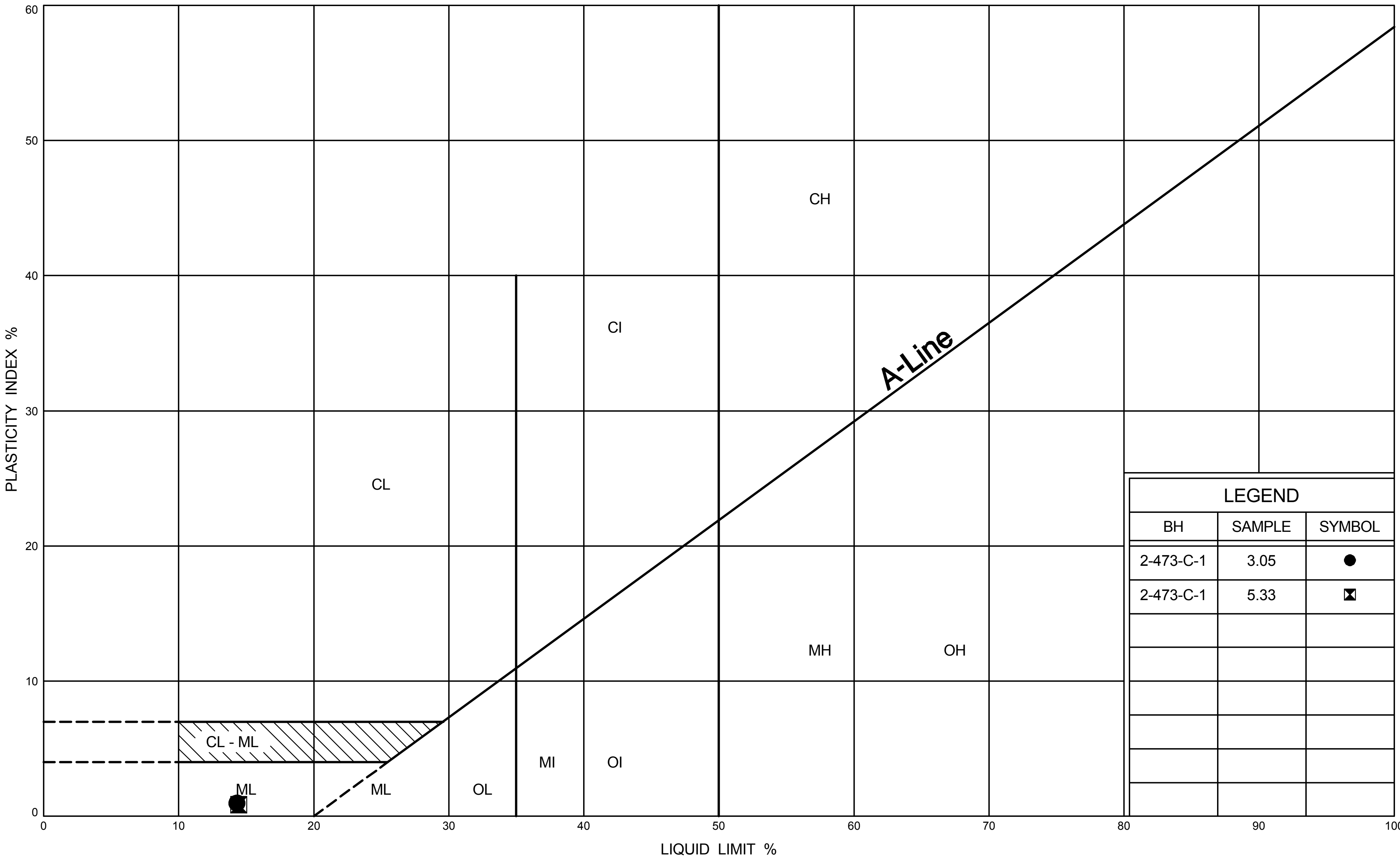
SAND, SILT AND SILT TILL (SP & ML)

FIG No 2

GWP 408-94-00

Highway 21-Kincardine to Tiverton

Oct 75, FF - S - 21



LEGEND		
BH	SAMPLE	SYMBOL
2-473-C-1	3.05	●
2-473-C-1	5.33	⧗

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PLASTICITY CHART
SILT & SILT TILL (ML)

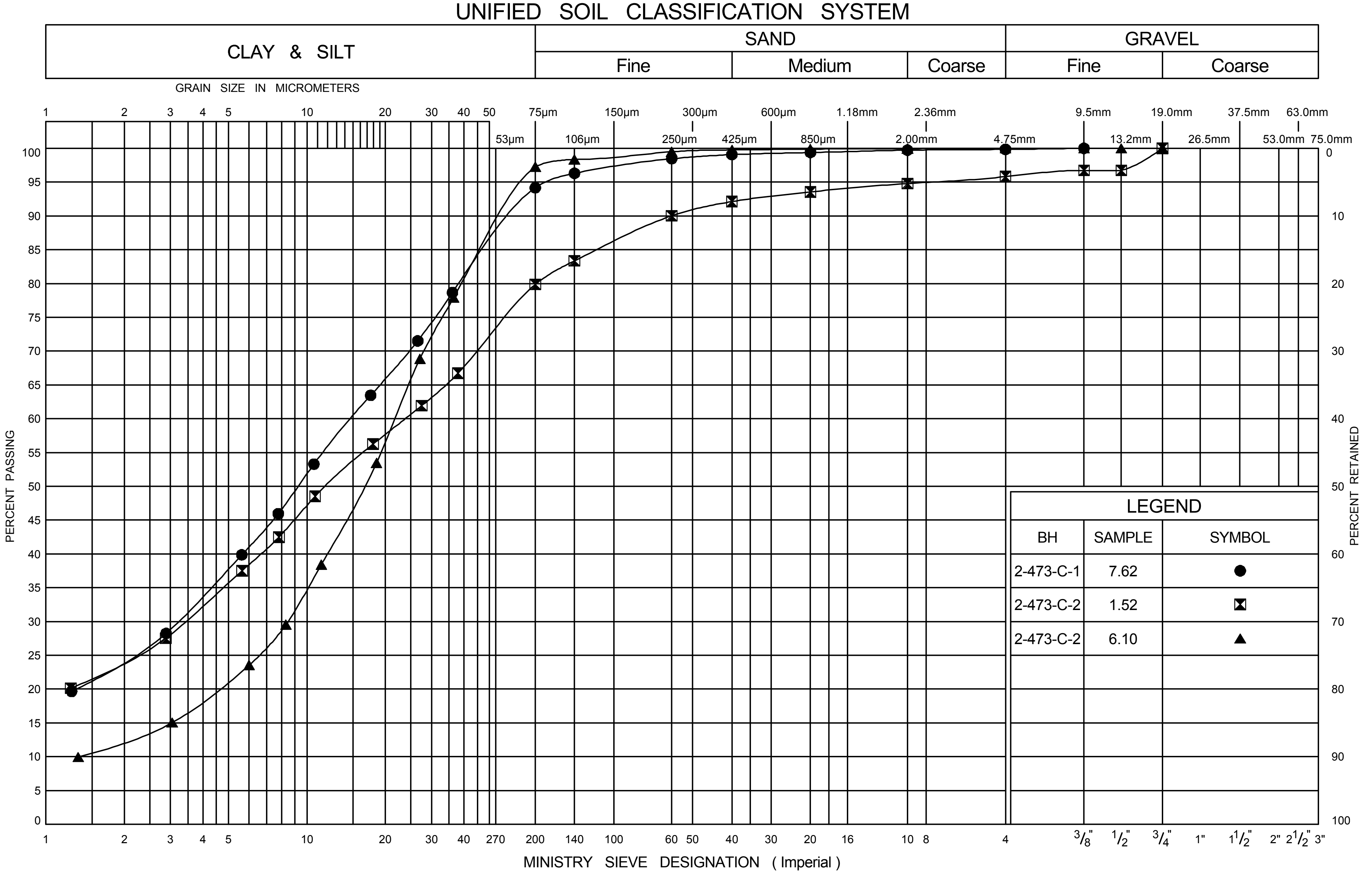
FIG No 3

GWP 408-94-00

Highway 21-Kincardine to Tiverton

ONTARIO MOT PLASTICITY CHART LARGE CULVE 06-8-IEG2.GPJ ONTARIO MOT.GDT 6/22/07

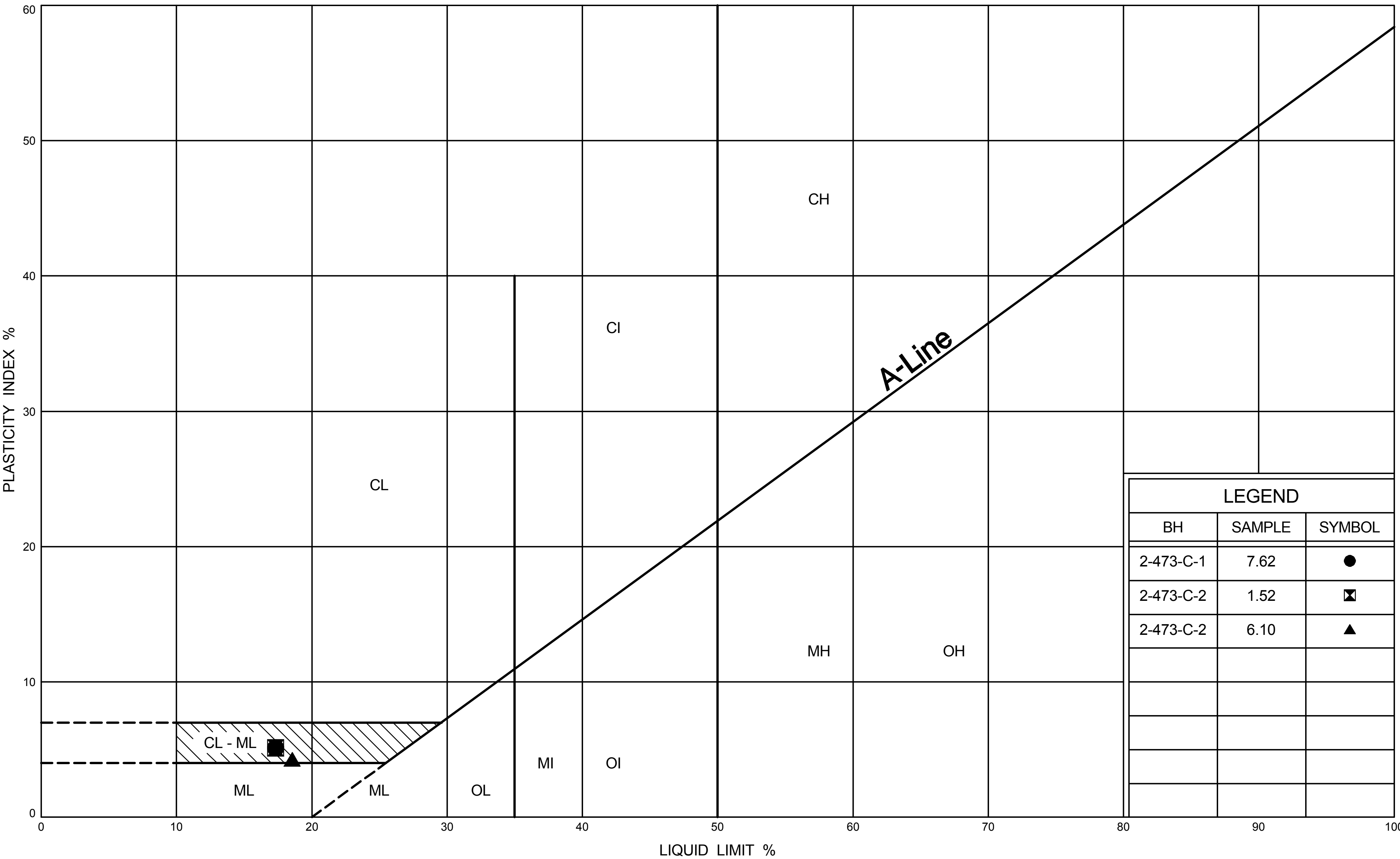
78 12 M



Ministry of Transportation Ontario	GRAIN SIZE DISTRIBUTION CLAYEY SILT LAYER (CL-ML)	FIG No 4 GWP 408-94-00 Highway 21-Kincardine to Tiverton

ONTARIO MOT GRAIN SIZE LARGE CULVERTS 06-8-IEG2.GPJ ONTARIO MOT.GDT 6/22/07

Oct 75, FF - S - 21



LEGEND		
BH	SAMPLE	SYMBOL
2-473-C-1	7.62	●
2-473-C-2	1.52	⌧
2-473-C-2	6.10	▲

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PLASTICITY CHART

CLAYEY SILT layer (CL-ML)

FIG No 5

GWP 408-94-00

Highway 21-Kincardine to Tiverton

ONTARIO MOT PLASTICITY CHART LARGE CULVE 06-8-IEG2.GPJ ONTARIO MOT.GDT 6/22/07

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.

Appendix D

Site Photographs

Culvert 2-473-C, General view

Culvert 2-473-C, Inlet

Culvert 2-473-C, Outlet