FOUNDATION INVESTIGATION
AND DESIGN REPORT

PROPOSED REPLACEMENT/EXTENSION OF
STRUCTURAL CULVERT 8-469C
OVER MEAFORD CREEK
HIGHWAY 26 FROM MEAFORD TO THORNBURY

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

I.E.
Group

MTO GEOCRES No. 41A-198

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PART A – FOUNDATION INVESTIGATION

1.0 INTRODUCTION

This report presents the results of a foundation investigation carried out in July and November 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 were not included in 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 8-469C over the Meaford Creek.

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. Since preparation of the draft foundation investigation and design report, the final culvert recommendations indicate that the culvert rehabilitation work for this structure will include:

- removal of a 9.7 m section of culvert and retaining walls on the inlet side and replaced with a 4.3 m section of culvert extension and associated wing walls and an armour stone retaining wall;
- reline 6.9 m length of the concrete arch section after repair of deteriorated concrete;
- some concrete floor repair within the arch section; and
- repair deteriorated concrete at the joint between the first and second section on the south side.

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 8-469C is located on Highway 26, approximately 2.1 km east of the east limit of the Town of Meaford, located at Station 25+314 over the Meaford Creek. Photographs of this culvert site are presented in Appendix “D”. The existing structure is a concrete arch rigid frame structure with non-rigid frame extensions. The span varies from 4.55 m to 6.4 m and the height varies from 3.7 to 4.5 m with an overfill height of 3.66 m to 4.5 m. The total length is 37.8 m with an additional 8.3 m of retaining walls extending south at the inlet and wing walls extending north at the outlet. The culvert is skewed at approximately 38 degrees to the roadway. The culvert opening dimensions were provided in the RFP documents.

The culvert spans over Meaford Creek which flows northerly. Meaford Creek incises a deep valley into the grey shale, which is exposed at the valley slopes at both upstream (south) and downstream (north) of the culvert. The valley slopes are standing at relatively steep inclinations which are estimated to be 1H: 1V to 1H:1.5V.

The approach embankments were built on both the east and west sides of the culvert, with a maximum height of approximately 8 to 12 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.

Grey shale is expected to form the streambed. There was approximately 1.0 m of water running in the creek at the time of the initial site visit during the proposal stage, and barely wet at the time of the field investigation.

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 231.50 m while the ground surface at the base of the embankment is some 8 to 12 m lower.

3.0 INVESTIGATION PROCEDURES

3.1 Field Investigation

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

On July 30 and 31, 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). 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. These undrained shear strengths are used to supplement the properties of the cohesive soils. It is noted that the measured shear strength value would be slightly lower than the actual value due to sampling disturbance. The soil samples obtained were placed in labeled containers and transported to IEG’s London laboratory for further examination and laboratory testing.

On November 2 and 3, 2007, a Diedrich D-50 Bombardier mounted drill rig was supplied by Walker Drilling Ltd. and used on site for obtaining rock core samples. Rock cores were retrieved using HQ core assembly (63 mm ID). The rock core samples were identified in the field and physical index properties were determined by visual examination and also by measurement of rock quality designations (RQD's) and rock core recovery. All rock cores were placed in wooden core boxes and transported to our laboratory for further examination, to confirm the field logging, and laboratory testing.

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 the purpose of proper management of the Borehole Logs within gINT, the borehole logging software, a preceding 0 was added to the culverts with a letter "A" or "B" also added after the culvert numbers to delineate Part A or Part B of this assignment. The boreholes were numbered 06A-1 to 06A-3 for the subject culvert and the depths of sampling were as follows:

Borehole No.	Depth of Sampling (m)
06A-1	6.25
06A-2	9.30
06A-3	18.29

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 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 and Atterberg Limit tests were performed on selected soil samples.

Two sections of the rock cores (at 13.7 m and 15.2 m depths) from Borehole 06A-3 were selected for unconfined compressive strength testing in accordance with ASTM D 2938. The testing was performed by Trow Associates Inc. of Brampton.

The results of the laboratory testing are presented on the Record of Borehole sheets (Appendix "A"), and Laboratory Test Results (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 dense embankment fill placed on a thin layer of very stiff to hard silty clay till which is in turn underlain by shale bedrock.

4.1.1 Pavement, Fill, Topsoil

Borehole 06A-2, which was located at the south edge of existing pavement in the shoulder area, encountered 760 mm shoulder gravel, a 100 mm thick layer of buried asphalt, and then 1.22 m of silty sand and gravel fill. At Boreholes 06A-1 and 06A-3, topsoil was contacted to depths of 0.15 m (elevation 226.45 m) and 0.05 m (elevation 230.58 m) respectively.

Underlying the shoulder gravel, asphalt and silty sand and gravel fill is the embankment fill material that extended to a depth of 4.42 m (elevation 227.08 m). The fill consists of brown silty clay with embedded sand and gravel. A single grain size distribution of the embankment fill is shown on Figure 1 of Appendix "B".

Standard penetration tests yielded "N"-values from 7 to 48 blows per 0.3 m. This fill is brown in colour and the measured natural moisture contents range from 5 to 22%. Based on the above field and laboratory test results, together and tactile examination, the fill materials exhibited loose to dense 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 Silty Clay Till

A stratum of grey silty clay till was contacted below the fill materials at Borehole 06A-2 and the topsoil layers at Borehole 06A-1 and 06A-3, and extended to depths of 0.91 to 5.49 m below the ground surface, at respective elevations of 224.47, 227.08 and 230.17 m. Two (2) grain size analyses were performed on the silty clay till deposit and the results are presented on Figure 2 of Appendix “B”.

Standard penetration tests yielded “N”-values from 17 to 30 blows per 0.3 m. 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)	34 and 38%, average at 36.0%
Plastic Limit (W_P)	20 and 21%, average at 20.5%
Plasticity Index (I_p)	14 and 17%, average at 15.5%

The natural moisture contents were in the range of 13 to 15%. These results are characteristic of clayey soils of low to medium plasticity (CL-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.

4.1.3 Shale Bedrock

The silty clay till was underlain by a stratum of grey shale of the Georgian Bay Formation. Grey, impure carbonate beds (limestone and dolomite, 10 to 200 mm thick layers) alternate with grey and blue/grey shale. The upper 1.5 to 2.2 m stratum, as noted in the boreholes, was slightly weathered, as revealed by core recovery of between 80 and 90% and RQD of between 60 and 100%. The underlying unweathered bedrock has recovery of between 95 and 100% and RQD of between 80 and 95%. The rock contains close to moderately close bedding planes which are typically flat as observed on the eroded valley banks.

The compressive strength of the shale can be described as week for the upper slightly weathered stratum, and medium strong for the underlying unweathered layer. Two unconfined compression tests on the rock core samples from Borehole 06A-3 (at 13.7 m and 15.2 m depths) yielded strengths of 29.8 and 36.0 MPa.

Two (2) grain size analysis was performed on the weathered shale and the results are presented on Figure 4 of Appendix “B”. Two (2) samples were tested and exhibited the following Atterberg Limit. These results are shown in Figure 5 of Appendix “B” and summarized below:

Liquid Limit (W_L)	37 and 39%, average at 38.0%
Plastic Limit (W_P)	20 and 21%, average at 20.5%
Plasticity Index (I_p)	17 and 18%, average at 17.5%

The natural moisture contents were in the typical range of 7 to 9%, indicative of damp moisture condition. These results are characteristic of medium plasticity (CI). A localized wet seam was encountered at 5.33 m depth in Borehole 06A-1, with a moisture content of 26%.

4.2 Groundwater Conditions

The groundwater condition was monitored during and upon completion of sampling. There was approximately 1.0 m of water running in the creek at the time of the initial site visit on March 14, 2007 (late Winter) during the proposal stage, and barely wet at the time of the field investigation on July 30 and 31 (Summer) and November 2 and 3 (Fall), 2007. The water levels observed in the creek likely reflected low flow conditions.

On completion of drilling, free groundwater was not observed in Borehole 06A-2. At Borehole 06A-1, the water level was 4.3 m (Elevation 222.30 m) on July 31, 2007 at the completion of drilling, with a localized wet seam was encountered at 5.33 m depth. At Borehole 06A-3, the groundwater observation could not be performed as water was used during the rock coring process, but the water level is expected to be close to the water level in the creek at approximately 12 m below ground surface at Elevation 219 m, on November 2 and 3, 2007.

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 replacement/extension of Structure 8-469C, 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 8-469C is located on Highway 26, approximately 2.1 km east of the east limit of the Town of Meaford, located at Station 25+314 over the Meaford Creek. Photographs of this culvert site are presented in Appendix "D". The existing structure is a concrete arch rigid frame structure with non-rigid frame extensions. The span varies from 4.55 m to 6.4 m and the height varies from 3.7 to 4.5 m with an overfill height of 3.66 m to 4.5 m. The total length is 37.8 m with an additional 8.3 m of retaining walls extending south at the inlet and wing walls extending north at the outlet. The culvert is skewed at approximately 38 degrees to the roadway. The culvert opening dimensions were provided in the RFP documents.

The culvert spans over Meaford Creek which flows northerly. Meaford Creek incises a deep valley into the grey shale which is exposed at the valley slopes at both upstream (south) and downstream (north) of the culvert. The valley slopes are standing at relatively steep inclinations which are estimated to be 1H : 1.0 to 1.5V.

The approach embankments were built on both the east and west sides of the culvert, with a maximum height of approximately 8 to 12 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.

Grey shale is expected to form the streambed. There was approximately 1.0 m of water running in the creek at the time of the initial site visit during the proposal stage, and barely wet at the time of the field investigation. The asphalt pavement surface over the existing culvert is near elevation 231.50 m while the ground surface at the base of the embankment is some 8 to 12 m lower.

Since preparation of the draft foundation investigation and design report, the final culvert recommendations indicate that the culvert rehabilitation work for this structure will include:

- removal of a 9.7 m section of existing culvert retaining walls on the inlet side and replaced with a 4.3 m section of culvert extension and associated wing walls, and an armour stone retaining wall;
- reline 6.9 m length of the concrete arch section after repair of deteriorated concrete;
- some concrete floor repair within the arch section; and
- repair deteriorated concrete at the joint between the first and second section on the south side.

The proposed culvert extension on the inlet (south) side could consist of either a precast concrete box culvert, a cast-in-place box culvert, or a rigid frame open-footing culvert.

5.2 Closed Box Culvert

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

The overburden soils and shale bedrock encountered at the subject site are considered suitable for the support of a box culvert foundation. Results of Boreholes 06A-1 and 06A-2 put down along the proposed culvert alignment indicate that the founding subgrade consists of slightly weathered to unweathered shale. The box culvert can be founded on the slightly weathered shale and designed at the following elevation and for bearing resistances shown below:

Highest Elevation (m)	Factored Geotechnical Resistance at ULS (kPa)	Geotechnical Reaction at SLS (kPa)
222.8	1,000	N/A

As the shale bedrock is a non-yielding foundation material, the ULS will govern the foundation design. This is based on the assumption that the shale bedrock subgrade will not be disturbed during excavation, and that the bedrock is protected from further weathering in the long term.

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 weathered shale could 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 the proposed extensions of the retaining walls, and designed to bear on the undisturbed slightly weathered shale and designed at the following elevation and for bearing resistances shown below:

Highest Elevation (m)	Factored Geotechnical Resistance at ULS (kPa)	Geotechnical Reaction at SLS (kPa)
222.8	1,000	N/A

As the shale bedrock is a non-yielding foundation material, the ULS will govern the foundation design. This is based on the assumption that the shale bedrock subgrade will not be disturbed during excavation, and that the bedrock is protected from further weathering in the long term.

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

Granular backfill should be constructed behind the culvert walls, headwalls (wing walls), and retaining walls as per OPSD-3121.150, with particular attention to the frost taper requirement. The granular backfill should conform to OPSS 1010 for either Granular "A" or Granular "B" Type III. To maintain free draining characteristics in granular fill materials, the maximum percentage passing the No. 200 sieve (75 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/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"** **$\phi = 30$ 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, weathered shale can be calculated using a coefficient of friction (friction factor) of 0.5 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 Retained Soil System (RSS)

A 4 m high Retained Soils System (RSS) may be used at the end of the new east wing walls wall to keep the toe of the embankment out of the creek for the final grade. The supplier of the RSS should be responsible for design of the structure such as backfill, reinforcement, and internal and external stability. The final culvert recommendations provided by Stantec indicated that the RSS system will consist of an armour stone wall. Details of the armour stone wall have not been finalized at the time of preparing this report. The following information should be included in the contract drawing:

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

Foundations of the armour stone wall will likely be placed on weathered shale bedrock at approximately Elevations 222.8 m. The armour stone wall can be designed to bear on the undisturbed slightly weathered shale and at the following elevation and for bearing resistances shown below:

Highest Elevation (m)	Factored Geotechnical Resistance at ULS (kPa)	Geotechnical Reaction at SLS (kPa)
222.8	1,000	N/A

As the shale bedrock is a non-yielding foundation material, the ULS will govern the foundation design. This is based on the assumption that the shale bedrock subgrade will not be disturbed during excavation, and that the bedrock is protected from further weathering in the long term.

The sliding resistance of the armour stone wall should be checked. It is assumed that a Granular A bedding layer, compacted to a minimum of 98% of the material's standard Proctor maximum dry density (SPMDD), will be placed beneath the armour stone wall. The unfactored horizontal resistance (Clause 6.7.5, CAN/CSA-S6-06) against sliding between the armour stone, the compacted Granular A bedding, and undisturbed, weathered shale can be calculated using a coefficient of friction (friction factor) of 0.4 as per Table 24.4 CFEM 4th Edition, 2006.

The backfill to be used for embankment construction will likely consist of imported, free draining Granular B material with soil unit weights and earth pressure coefficients provided in Section 5.4.

5.6 Embankment Widening

The existing approach embankments are up to 12 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.

Due to the height of the embankment fill of greater than 8 m, a 1 m 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 native silty clay till or weathered shale are anticipated for embankment slope of 2.5H:1V and up to 12 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.

5.7 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 8 and 12 m.

Excavation to depths of up to 12 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 subgrade 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 materials 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.

Excavation within the shale bedrock can be carried out with near vertical side slopes. It is noted that limestone and dolomite layers (10 to 200 mm thick) are present within the shale and could present some difficulties. Excavations through the shale bedrock will require a large excavator equipped with hoe-ram.

Temporary support within the overfill of the existing and the new partially constructed 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.

5.8 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.

5.9 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. Silty clay till and shale could be exposed at the streambed, and their permissible velocities[1] are 1.8 m/s and 3.0 m/s respectively.

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

6.0 STATEMENT OF LIMITATION

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

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

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

Yours truly,
Infrastructure Engineering Group Inc.

LICENSED PROFESSIONAL ENGINEER
E.Y. CHUNG
8345506
Dec 17-08
PROVINCE OF ONTARIO

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

Joseph Law, P.Eng.
Project Manager

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

LICENSED PROFESSIONAL ENGINEER
J. L.
J. C-Y. LAW
12/17/08
PROVINCE OF ONTARIO

Drawing 1

Borehole Locations

And

Soil Strata

METRIC

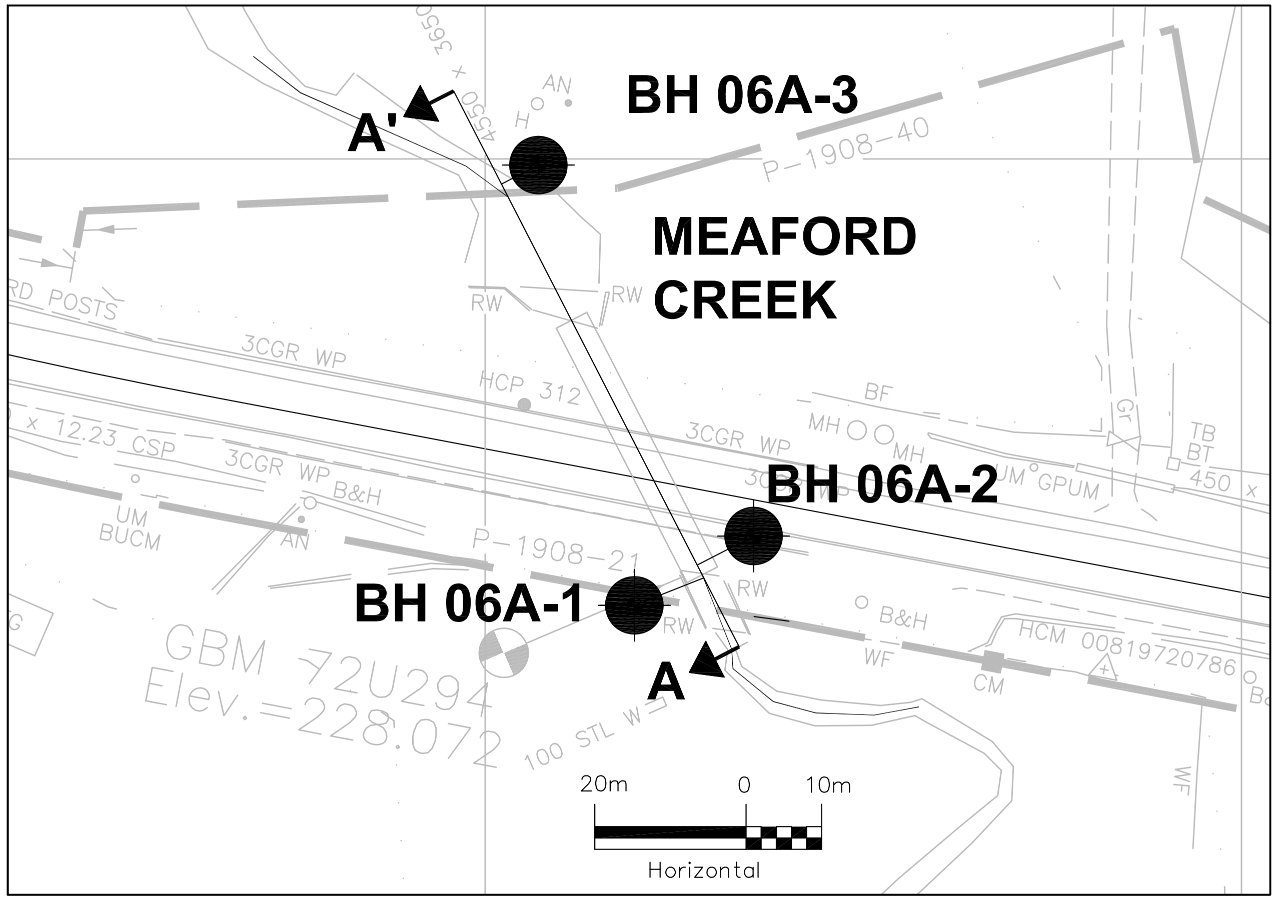
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN

CONT No xxxx-xxxx
WP No GWP 57-00-00

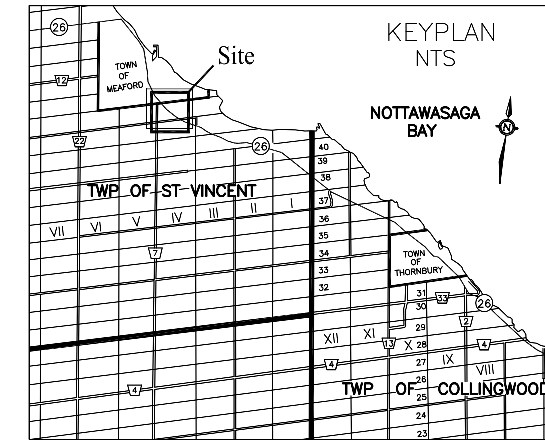
Culvert # 8-469C
Meaford Creek, Highway 26
BORE HOLE LOCATIONS & SOIL STRATA

SHEET 1

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

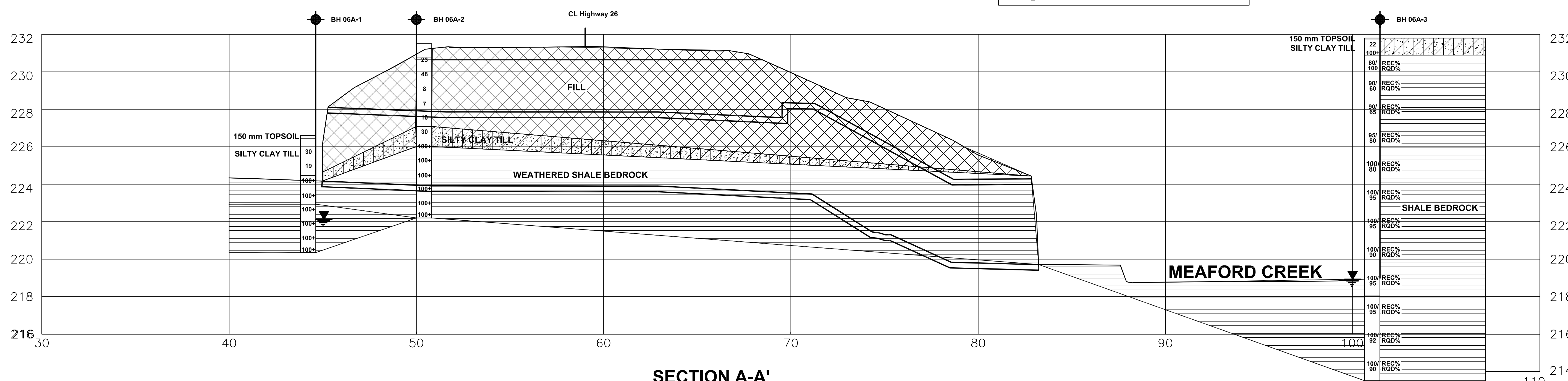


BOREHOLE LOCATION PLAN



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



SECTION A-A'
CENTERLINE OF CULVERT

REVISIONS	DATE	BY	DISCRIPTION
	08/12/08	J.L.	Final
	27/02/08	J.L.	Draft

Geocres : 41A-198

BOREHOLE NO.	ELEVATION	UTM CO-ORDINATES NORTH	EAST
06A-1	226.60	4939241	220820
06A-2	231.50	4939250	220835
06A-3	231.08	4939317	220737

HWY No. HWY 26		DIST	Owen Sound
SUBM'D J.L.	CHECKED E.C.	DATE 25/01/08	SITE 8-469C
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.

Appendix A

Explanation of Terms Used in Report

Record of Borehole Sheet

Boreholes 06A-1 to 06A-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 'A' SIZE DRILL RODS. THE RESISTANCE TO CONE PENETRATION IS MEASURED AS THE NUMBER OF BLOWS FOR EACH 0.3m ADVANCE OF THE CONICAL POINT INTO THE UNDISTURBED GROUND.

SOILS ARE DESCRIBED BY THEIR COMPOSITION AND CONSISTENCY OR DENSENESS.

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

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

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

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

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

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

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

RQD (%)	0 - 25	25 - 50	50 - 75	75 - 90	90 - 100
	VERY POOR	*POOR*	*FAIR*	*GOOD*	*EXCELLENT*

JOINTING AND BEDDING:

SPACING	50mm	50 - 300mm	0.3m - 1m	1m - 3m	>3m
JOINTING	*VERY CLOSE*	*CLOSE*	*MOD. CLOSE*	*WIDE*	*VERY WIDE*
BEDDING	*VERY THIN*	*THIN*	*MEDIUM*	*THICK*	*VERY THICK*

ABBREVIATIONS AND SYMBOLS

FIELD SAMPLING

S S	SPLIT SPOON	T P	THINWALL PISTON
W S	WASH SAMPLE	O S	OSTERBERG SAMPLE
S T	SLOTTED TUBE SAMPLE	R C	ROCK CORE
B S	BLOCK SAMPLE	P H	T W ADVANCED HYDRAULICALLY
C S	CHUNK SAMPLE	P M	T W ADVANCED MANUALLY
T W	THINWALL OPEN	F S	FOIL SAMPLE

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

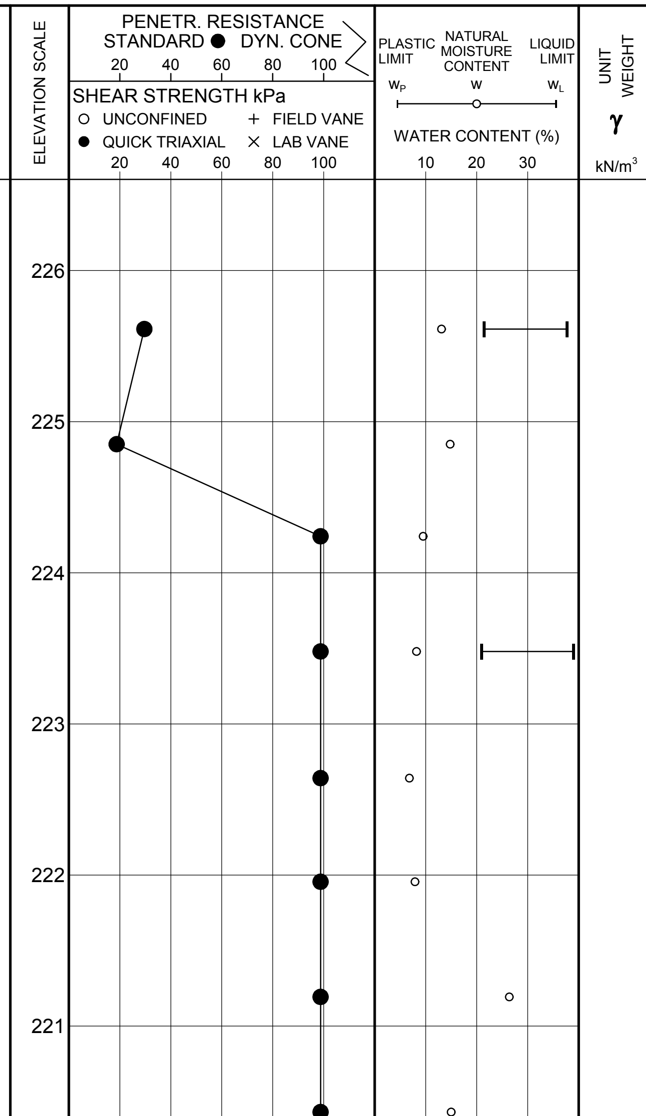
e_{min}	1,%	VOID RATIO IN DENSEST STATE
I_D	1	DENSITY INDEX $= \frac{e_{max} - e}{e_{max} - e_{min}}$
D	mm	GRAIN DIAMETER
D_n	mm	n PERCENT - DIAMETER
C_U	1	UNIFORMITY COEFFICIENT
h	m	HYDRAULIC HEAD OR POTENTIAL
q	m^3/s	RATE OF DISCHARGE
v	m/s	DISCHARGE VELOCITY
i	1	HYDRAULIC GRADIENT
k	m/s	HYDRAULIC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

RECORD OF BOREHOLE No 06A-1

METRIC

W.P.	GWP 57-00-00	LOCATION	Northing - 4939241, Easting - 220820
DIST	Owen Sound HWY 26	BOREHOLE TYPE	S/S Augering, 110 mm dia.
DATUM	Geodetic	DATE	07.30.07 - 07.30.07
ORIGINATED BY	JL	COMPILED BY	JL
CHECKED BY	EC		

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
226.60	Ground					
0.00	150 mm TOPSOIL.					
	Silty CLAY TILL, CI Grey, moist, hard to very stiff, with embedded sand and gravel, shale fragments.	1	SPT	30		25 15 37 23 (61)
		2	SPT	19		
224.47 2.13	SHALE BEDROCK Grey, weathered, weak to medium strong, close to moderately close bedding, fair quality, occ. limestone layers (10 to 20mm thick).	3	SPT	100+		
		4	SPT	100+		4 8 50 38 (88)
		5	SPT	100+	∇	Water level measured @ 4.3m @ completion.
		6	SPT	100+		
		7	SPT	100+		Wet seam @ 5.33 m.
220.35		8	SPT	100+		
6.25	End of borehole.					



+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 06A-2

METRIC

W.P. GWP 57-00-00	LOCATION Northing - 4939250, Easting - 220835		ORIGINATED BY JL
DIST Owen Sound	HWY 26	BOREHOLE TYPE S/S Augering, 110 mm dia.	COMPILED BY JL
DATUM Geodetic		DATE 07.31.07 - 07.31.07	CHECKED BY EC

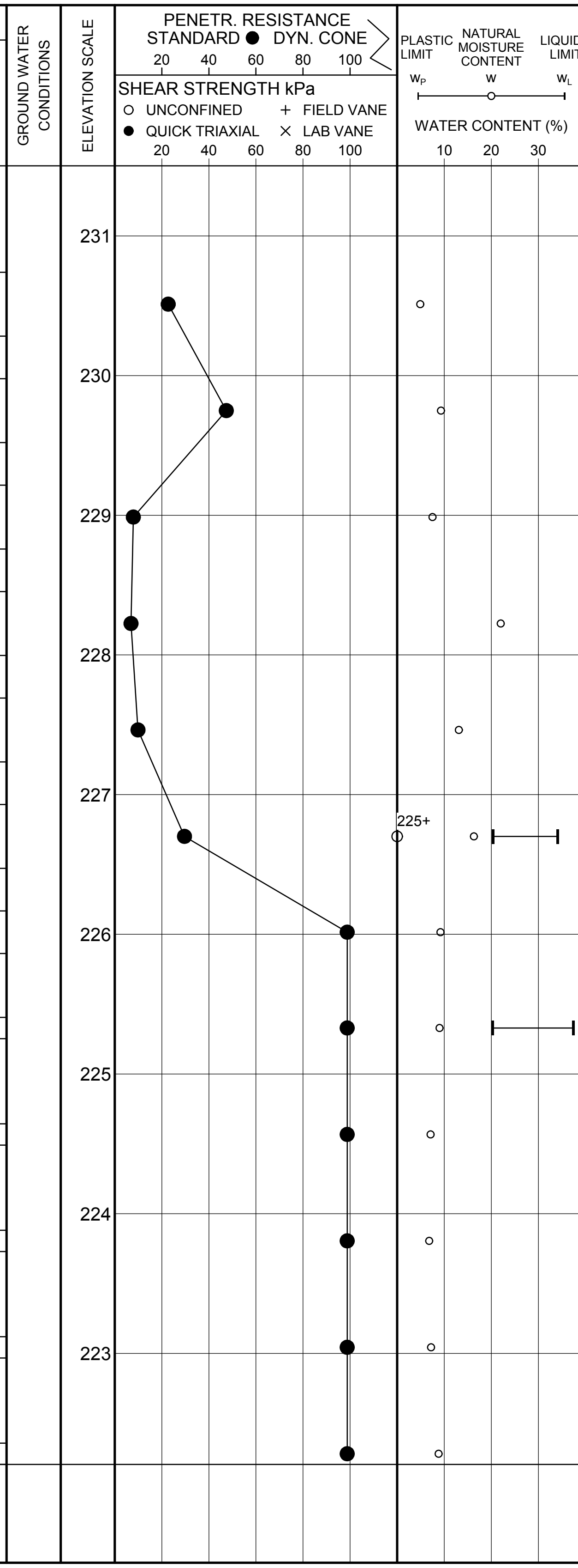
SOIL PROFILE

ELEV / DEPTH	DESCRIPTION
231.50	Ground
0.00	FILL Brown, moist, compact, consisting mainly of sand and gravel, 40mm stones.
230.74 / 0.76	100 mm ASPHALT FILL Brown, moist, compact to dense, consisting of silty sand amnd gravel.
229.52 / 1.98	FILL Brown, moist to wet, loose to compact, consisting of silty clay with embedded sand and gravel.
227.23 / 4.27	Silty CLAY TILL, CL Brown Changing to grey, moist to wet, firm to hard, with embedded sand and gravel, shale fragments.
226.01 / 5.49	SHALE BEDROCK Grey, slightly weathered, weak, close to moderately close bedding, fair quality, occ. limestone layers (10 to 20mm thick).
224.64 / 6.86	SHALE BEDROCK Grey, unweathered, medium strong, close to moderately close bedding, fair quality, occ. limestone layers (10 to 20mm thick).
222.20 / 9.30	End of borehole.

SAMPLES

NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
1	SPT	23	
2	SPT	48	41 38 15 6 (22)
3	SPT	8	
4	SPT	7	
5	SPT	10	
6	SPT	30	6 8 55 31 (87)
7	SPT	100+	
8	SPT	100+	8 16 42 33 (75)
9	SPT	100+	
10	SPT	100+	
11	SPT	100+	
12	SPT	100+	

Borehole dry and open @ completion.



UNIT WEIGHT γ kN/m³

GROUND WATER CONDITIONS

$+^3, \times^3$: Numbers refer to Sensitivity

$\bigcirc^{150}$ UNCONFINE SHEAR STRENGTH INFERRED FROM POCKET PENETROMETER READINGS

RECORD OF BOREHOLE No 06A-3

METRIC

W.P. GWP 57-00-00 | LOCATION Northing - 4939317, Easting - 220737 | ORIGINATED BY JL

DIST Owen Sound HWY 26 | BOREHOLE TYPE HQWL, 63.5 mm ID | COMPILED BY JL

DATUM Geodetic | DATE 11.02.07 - 11.03.07 | CHECKED BY EC

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%)
231.08	Ground				
0.00	50 mm TOPSOIL.				
	Silty CLAY TILL (CI) Brown, moist, very stiff to hard, embedded sand and gravel.	1	SPT	22	
		2	SPT	100+	
230.17 / 0.91	SHALE BEDROCK Grey, weathered, weak to medium strong, close to moderately close bedding, fair quality, occ. limestone layers (10 to 20mm thick).	RC3	NQ		Recovery - 80%, RQD - 100%
		RC4	NQ		Recovery - 90%, RQD - 60%
		RC5	NQ		Recovery - 90%, RQD - 65%
226.97 / 4.11	SHALE BEDROCK Grey, unweathered, medium strong, close to moderately close bedding, good quality, limestone layers (15 to 150mm thick).	RC6	NQ		Recovery - 95%, RQD - 80%
224.98 / 6.10	SHALE BEDROCK Grey, unweathered, medium strong, excellent quality.	RC7	NQ		Recovery - 100%, RQD - 80%
		RC8	NQ		Recovery - 100%, RQD - 95%
		RC9	NQ		Recovery - 100%, RQD -

Chart headings: 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); PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 10 20 30; UNIT WEIGHT γ kN/m³; GR SA SI CL; GROUND WATER CONDITIONS; ELEVATION SCALE 231–222.

Continued Next Page

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

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Ontario

RECORD OF BOREHOLE No 06A-3

METRIC

W.P. GWP 57-00-00 LOCATION Northing - 4939317, Easting - 220737 ORIGINATED BY JL

DIST Owen Sound HWY 26 BOREHOLE TYPE HQWL, 63.5 mm ID COMPILED BY JL

DATUM Geodetic DATE 11.02.07 - 11.03.07 CHECKED BY EC

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	SHALE BEDROCK Grey, unweathered, medium strong, excellent quality. *(continued)*				221	95%
		RC10	NQ		220	Recovery - 100%, RQD - 90%
		RC11	NQ		219, 218	Recovery - 100%, RQD - 95%
		RC12	NQ		217	Uniaxial Compressive Strength = 29.8 MPa; Recovery - 100%, RQD - 95%
		RC13	NQ		216, 215	Uniaxial Compressive Strength = 36.0 MPa; Recovery - 100%, RQD - 92%
212.79		RC14	NQ		214, 213	Recovery - 100%, RQD - 90%
18.29	End of borehole.					Water was used to aid diamond coring. Water level at ground surface during coring (return water). Water level assumed to be @ creek bottom @ 12m below the existing ground surface.

Column headings: SOIL PROFILE; SAMPLES; STRAT PLOT; GROUND WATER CONDITIONS; 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; PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L WATER CONTENT (%) 10 20 30; UNIT WEIGHT γ kN/m^3

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

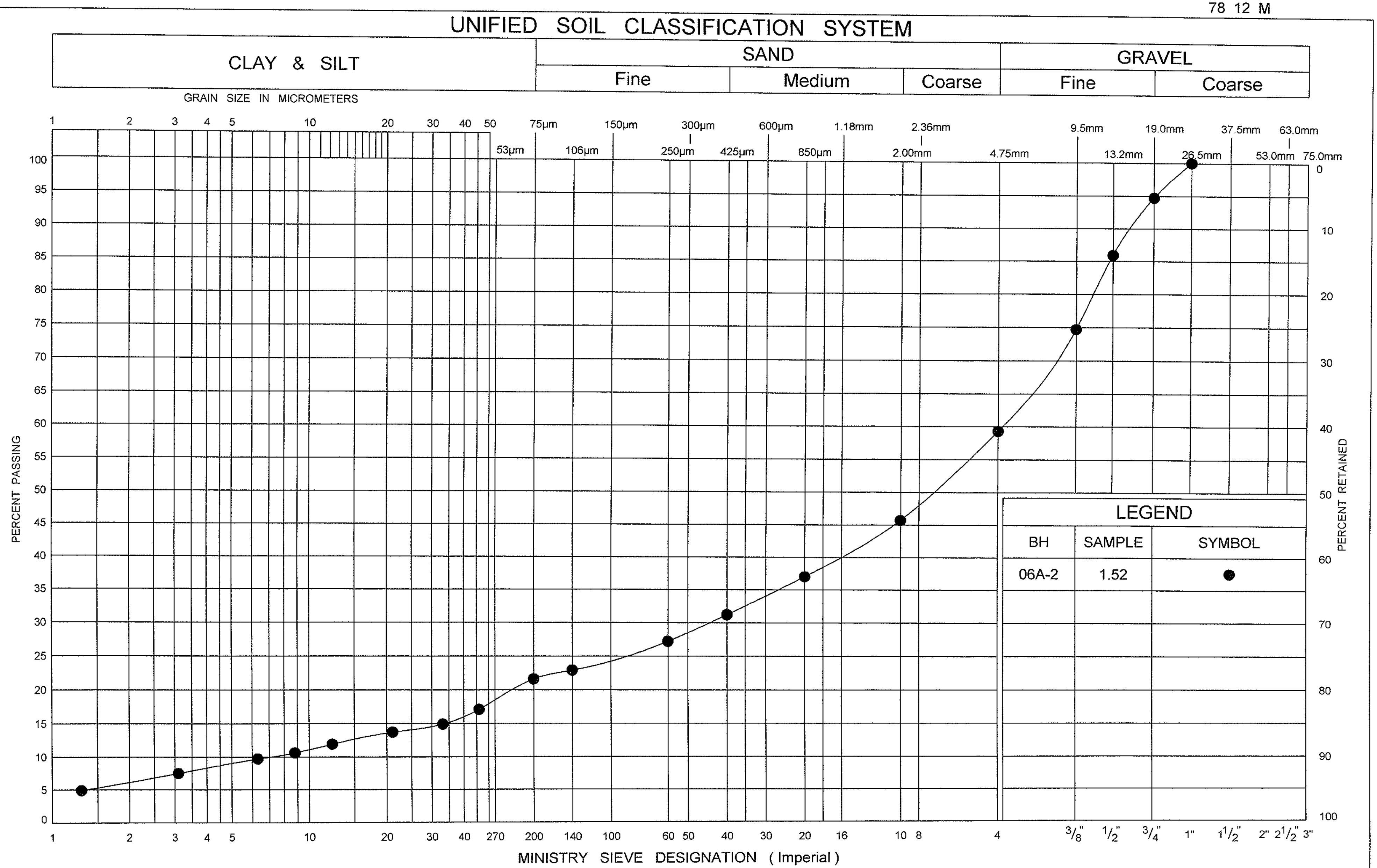
Appendix B

Laboratory Test Results

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

Rock Core Compression Report by Trow Associates Inc.

78 12 M



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GRAIN SIZE DISTRIBUTION

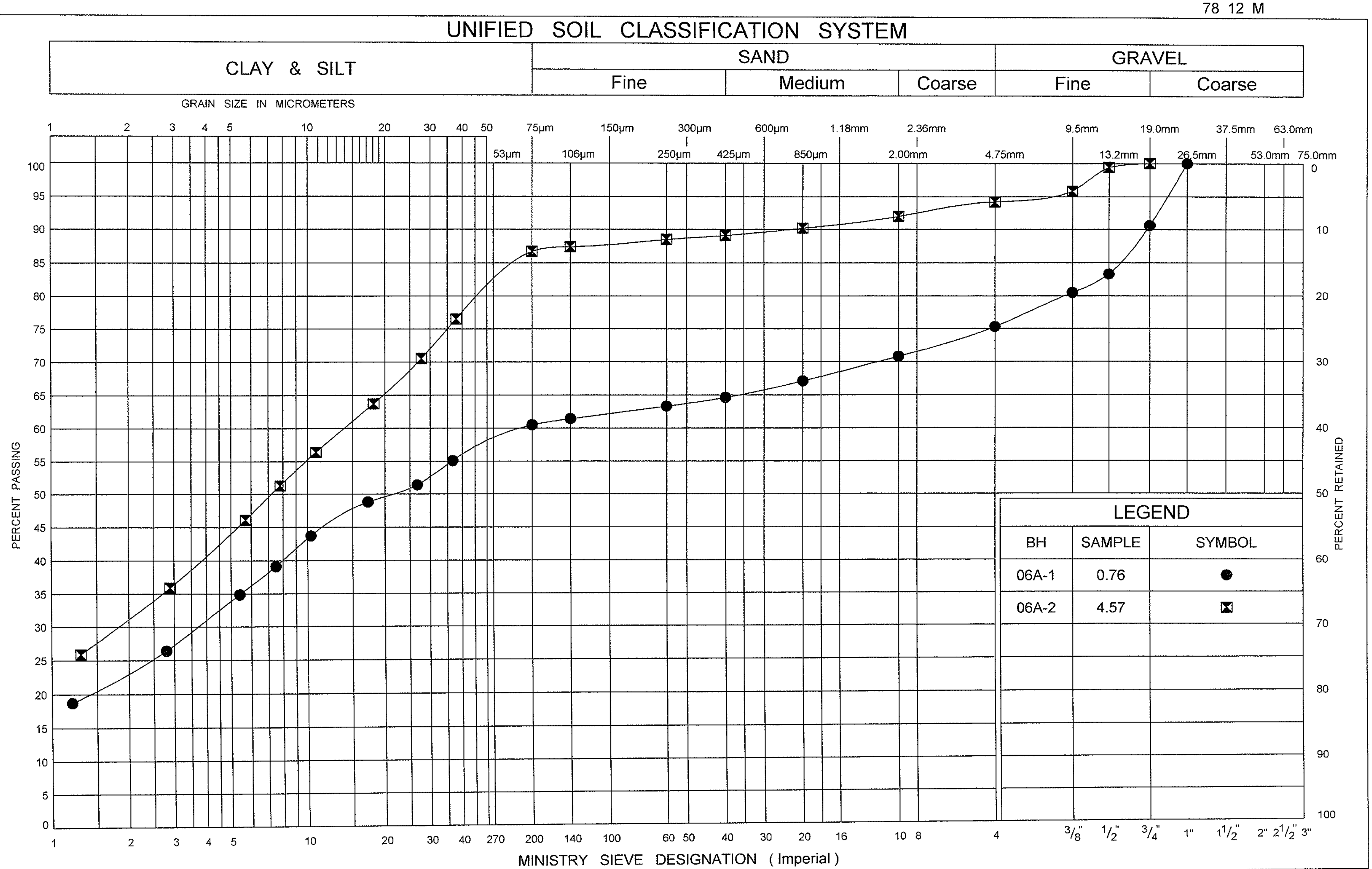
FILL

FIG No 1

GWP 57-00-00

HWY 26, Thornbuy to Meaford

78 12 M



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GRAIN SIZE DISTRIBUTION

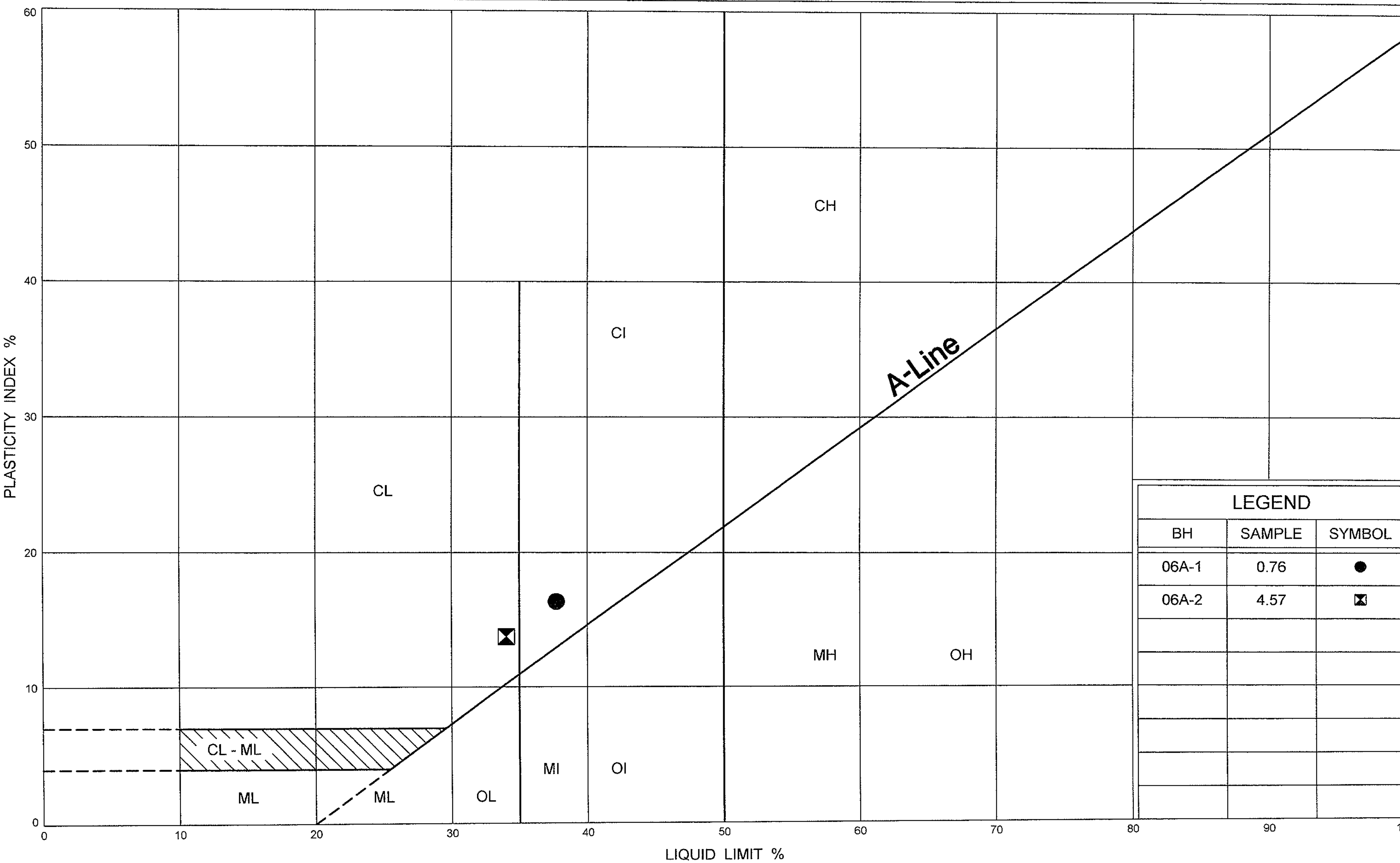
SILTY CLAY TILL

FIG No 2

GWP 57-00-00

HWY 26, Thornbury to Meaford

Oct 75, FF - S - 21



LEGEND		
BH	SAMPLE	SYMBOL
06A-1	0.76	●
06A-2	4.57	⊠

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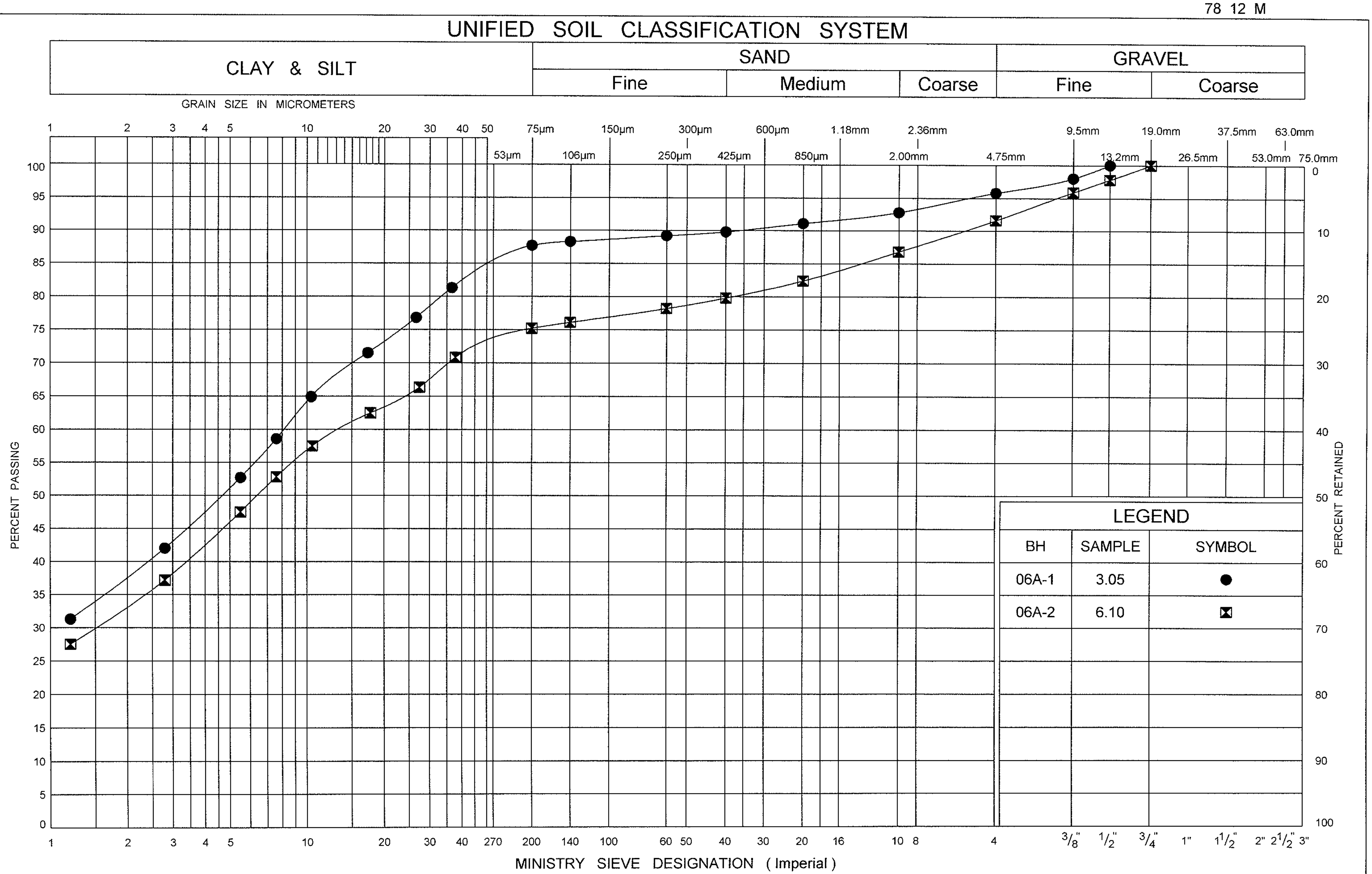
PLASTICITY CHART
SILTY CLAY TILL

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

78 12 M



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GRAIN SIZE DISTRIBUTION

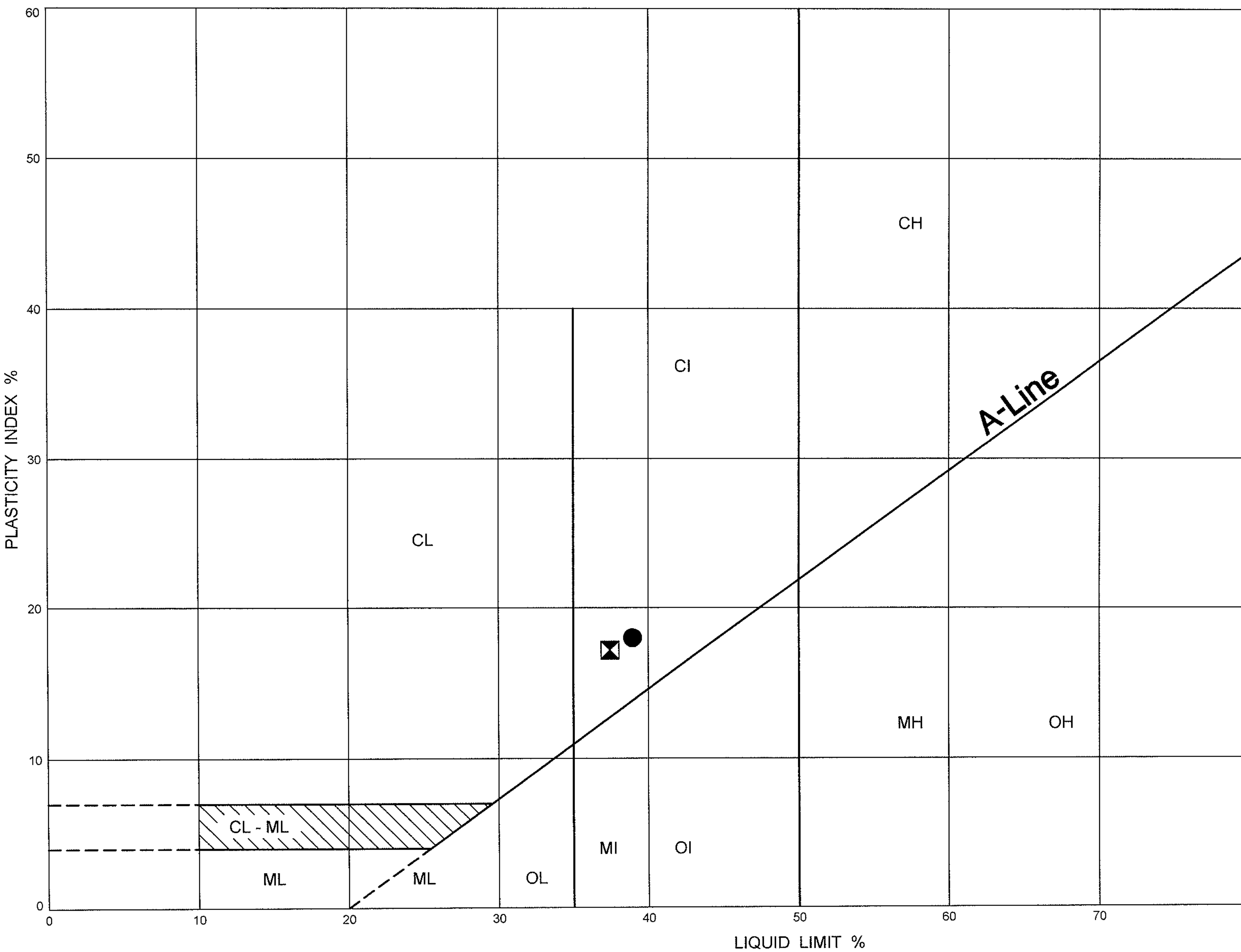
WEATHERED SHALE

FIG No 4

GWP 57-00-00

HWY 26, Thornbuy to Meaford

Oct 75, FF - S - 21



LEGEND		
BH	SAMPLE	SYMBOL
06A-1	3.05	●
06A-2	6.10	⧖

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Ontario

PLASTICITY CHART
WEATHERED SHALE

FIG No 5

GWP 57-00-00

HWY 26, Thornbuy to Meaford

Trow
50 YEARS
Since 1957

1595 Clark Boulevard
Brampton, Ontario
L6T 4V1

Tel: 905-793-9800
Fax: 905-793-0641

Buildings

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Geotechnical

Infrastructure

Materials & Quality

FACSIMILE

Date: December 10, 2007

Facsimile No.: 1-519-680-9993 **Reference No.: LAGM00289085B**

Attention: Mr. Joe Law, P.Eng. Law Engineering

cc:

From: Mr. Ammanuel Yousif

Dept: Geotechnical Lab

Operator: Ausenda Meco **Total Pgs. (including this one)** 2

Subject: *Concrete Core Test Report for Hwy 26, Meaford Bridge*

☐ Urgent ☐ For Review ☐ Please Reply ☐ Please Distribute

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ROCK CORE Test Report

Project No.: LAGM00289085B **Project Name:** Hwy 26, Meaford Bridge

Core No.	BH-06A-3	BH-06A-3
Location	45'	50'
Date Cored		
Date Tested	November 29. 2007	November 29, 2007
Height - (mm)	180.0	164.0
Average Diameter - (mm)	63.0	63.1
Corrected Compressive Strength - (MPa)	29.8	36.0

Tests in accordance with C.S.A. CAN-A23.2-14C, unless otherwise indicated.
NOTE: Relative to direction of compaction of concrete when placed.

Testing Laboratory Representative Signature

Nov-29/07
Date

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

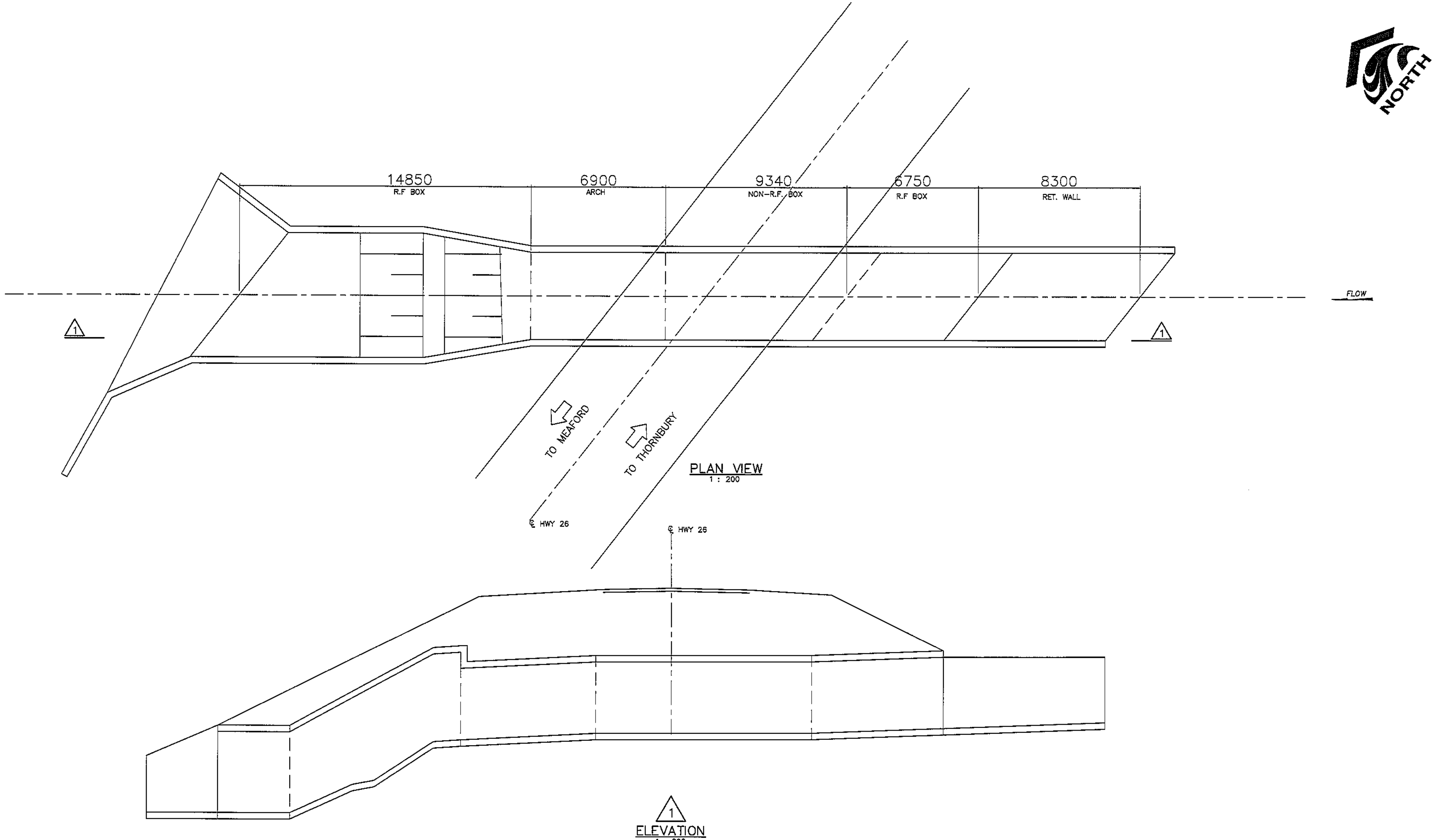
Site Photographs

Station 25+314 – Downstream end (Meaford Creek)

Station 25+314 – Downstream end (Meaford Creek)

Station 25+314 – Looking upstream (south)

Station 25+314 – Upstream end (south)



HIGHWAY 26 THORNBURY TO MEAFORD W.P. 57-00-00	MEAFORD CREEK CULVERT SITE 8-469C Ministry of Transportation Ontario Stantec	CULVERT INSPECTION AND EVALUATION REPORT	FIGURE 1

Photo 1: General view of road, looking west.

Photo 2: South elevation.

Highway 26
W.P. 57-00-00
Site No. 8-469C
June 1, 2005

Photo 3: North elevation.

Photo 4: Looking downstream.

Photo 5: Looking south through culvert.

Photo 6: Looking north through culvert.

Highway 26 Site No. 8-469C June 1, 2005
W.P. 57-00-00

Photo 7: Undermining at outlet.

Photo 8: Cracking and leaching at north headwall.

Photo 9:

Typical construction joint at south extension.

Photo 10:

Delamination and wetness at soffit construction joint (south extension).

Highway 26 Site No. 8-469C June 1, 2005
W.P. 57-00-00

Photo 11: View of arch, looking north.

Photo 12: View of arch, looking south.

Photo 13: Delaminations and cracks in arch, east wall.

Photo 14: Floor slab failure at arch, looking southwest.

Highway 26 Site No. 8-469C June 1, 2005
W.P. 57-00-00

Photo 15:

Floor slab failure at arch, looking north.

Photo 16:

View of arch soffit, looking north.