

FOUNDATION INVESTIGATION 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

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Prepared for:

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07-6-IEG1-8-469C

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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 weak 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.

5.0 STATEMENT OF LIMITATION

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

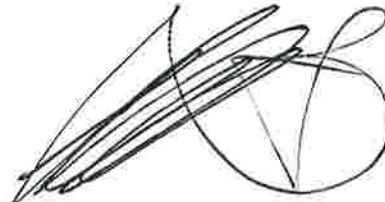
The Limitations of Report, as Quoted in Appendix "C", is an integral part of this report.


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

Yours truly,
Infrastructure Engineering Group Inc.


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




Joseph Law, P.Eng.
Project Manager


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



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

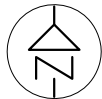
07-6-IEG1-8-469C
Final Report
Drawing 1
December 17, 2008

Drawing 1
Borehole Locations
And
Soil Strata

METRIC

DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN

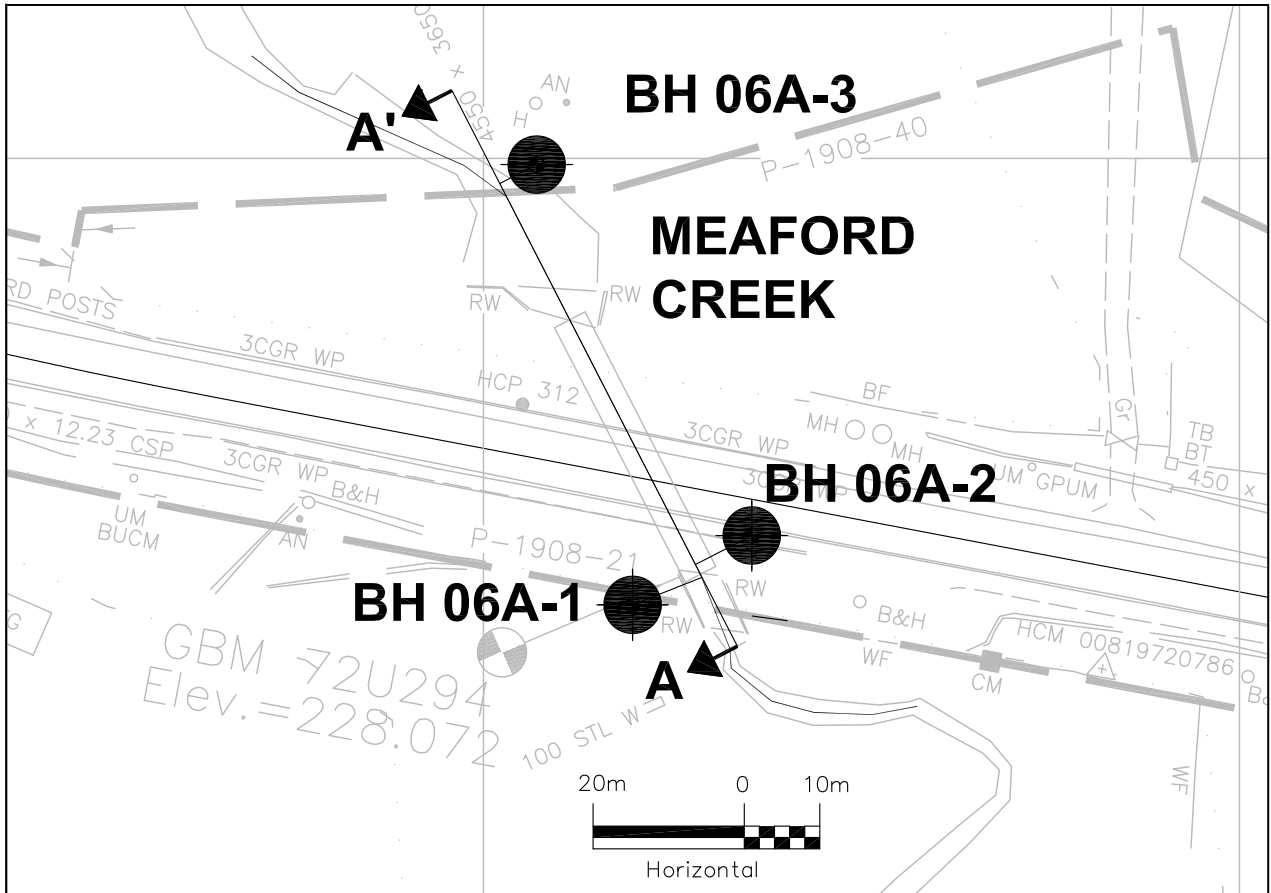
CONT No xxxx-xxxx
WP No GWP 57-00-00



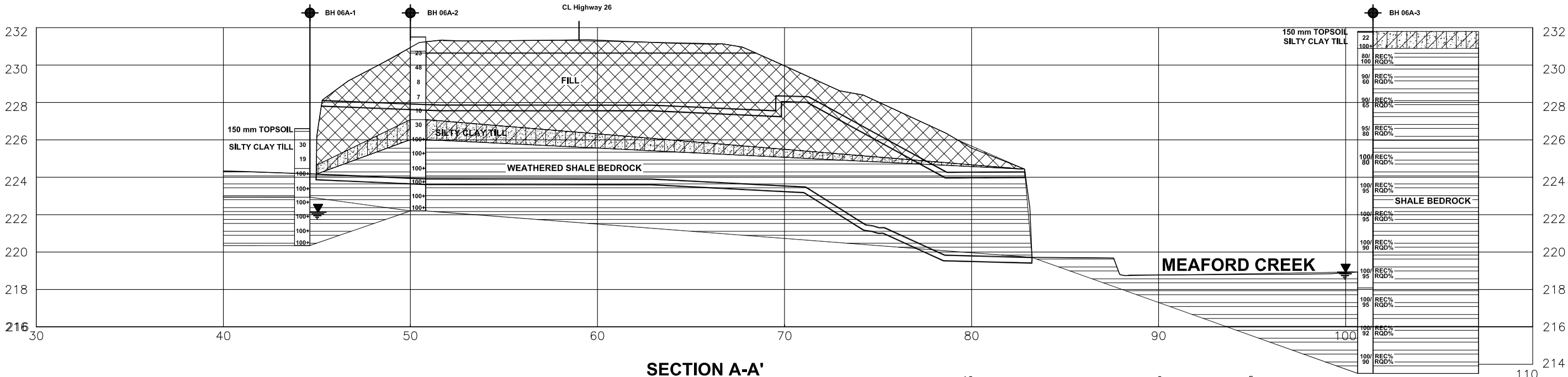
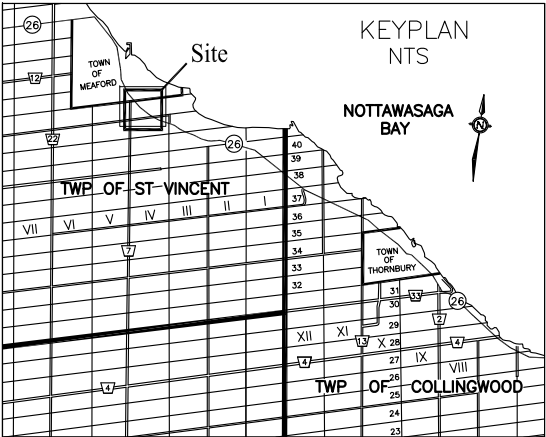
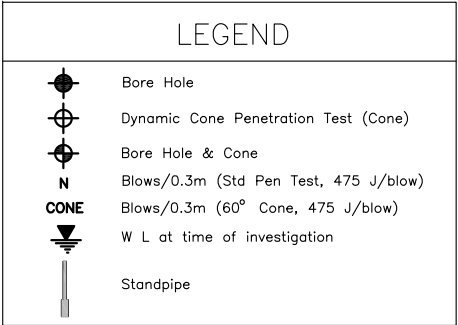
Culvert # 8-469C
Meaford Creek, Highway 26
BORE HOLE LOCATIONS & SOIL STRATA

SHEET
1

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



BOREHOLE LOCATION PLAN



SECTION A-A'
CENTERLINE OF CULVERT

NOTES

- THE COMPLETE FOUNDATION INVESTIGATION AND DESIGN REPORT FOR THIS PROJECT AND OTHER RELATED DOCUMENTS MAY BE EXAMINED AT THE ENGINEERING MATERIALS OFFICE, DOWNSVIEW. INFORMATION CONTAINED IN THIS REPORT AND RELATED DOCUMENTS ARE SPECIFICALLY EXCLUDED IN ACCORDANCE WITH THE CONDITIONS OF SECTION GC2.01 of OPS GEN. COND.
- THE BOUNDARIES BETWEEN SOIL STRATA HAVE BEEN ESTABLISHED ONLY AT BOREHOLE LOCATIONS. BETWEEN BOREHOLES AND BOUNDARIES ARE ASSUMED FROM GEOLOGICAL EVIDENCE.
- SUBGRADE ELEVATION OF THE EXISTING FOOTING NOT KNOWN AND IS ESTIMATED TO BE AT 1.2m BELOW THE CREEK BED.
- THIS DRAWING IS FOR SUBSURFACE INFORMATION ONLY. SURFACE DETAILS AND FEATURES ARE FOR CONCEPTUAL ILLUSTRATION.

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

REVISIONS			
	08/12/08	J.L.	Final
	27/02/08	J.L.	Draft
	DATE	BY	DISCRIPTION
Geocres : 41A-198			
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

Ministry of Transportation/Stantec Consulting Ltd.
G.W.P. 57-00-00
Rehabilitation of Highway 26 from Meaford to Thornbury
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07-6-IEG1-8-469C
Final Report
Appendix A
December 17, 2008

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 1" SIZE DRILL RODS. THE RESISTANCE TO CONE PENETRATION IS MEASURED AS THE NUMBER OF BLOWS FOR EACH 0.3m ADVANCE OF THE CONICAL POINT INTO THE UNDISTURBED GROUND.

SOILS ARE DESCRIBED BY THEIR COMPOSITION AND CONSISTENCY OR DENSENESS.

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

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

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

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

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

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

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

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

JOINTING AND BEDDING:

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

ABBREVIATIONS AND SYMBOLS

FIELD SAMPLING

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

STRESS AND STRAIN

u_w	kPa	PORE WATER PRESSURE
r_u	1	PORE PRESSURE RATIO
σ	kPa	TOTAL NORMAL STRESS
σ'	kPa	EFFECTIVE NORMAL STRESS
τ	kPa	SHEAR STRESS
$\sigma_1, \sigma_2, \sigma_3$	kPa	PRINCIPAL STRESSES
ϵ	%	LINEAR STRAIN
$\epsilon_1, \epsilon_2, \epsilon_3$	%	PRINCIPAL STRAINS
E	kPa	MODULUS OF LINEAR DEFORMATION
G	kPa	MODULUS OF SHEAR DEFORMATION
μ	1	COEFFICIENT OF FRICTION

MECHANICAL PROPERTIES OF SOIL

m_v	kPa ⁻¹	COEFFICIENT OF VOLUME CHANGE
C_c	1	COMPRESSION INDEX
C_s	1	SWELLING INDEX
C_α	1	RATE OF SECONDARY CONSOLIDATION
C_v	m ² /s	COEFFICIENT OF CONSOLIDATION
H	m	DRAINAGE PATH
T_v	1	TIME FACTOR
U	%	DEGREE OF CONSOLIDATION
σ'_{vo}	kPa	EFFECTIVE OVERBURDEN PRESSURE
σ'_p	kPa	PRECONSOLIDATION PRESSURE
τ_f	kPa	SHEAR STRENGTH
c'	kPa	EFFECTIVE COHESION INTERCEPT
ϕ'	-°	EFFECTIVE ANGLE OF INTERNAL FRICTION
c_u	kPa	APPARENT COHESION INTERCEPT
ϕ_u	-°	APPARENT ANGLE OF INTERNAL FRICTION
τ_r	kPa	RESIDUAL SHEAR STRENGTH
τ_r	kPa	REMOULDED SHEAR STRENGTH
S_t	1	SENSITIVITY = $\frac{c_u}{\tau_r}$

PHYSICAL PROPERTIES OF SOIL

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

RECORD OF BOREHOLE No 06A-1

1 OF 1

METRIC

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

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	PENETR. RESISTANCE		PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			STANDARD ● DYN. CONE						
								SHEAR STRENGTH kPa						
226.60 0.00	Ground							20 40 60 80 100						
	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			3	SPT	100+									
	SHALE BEDROCK Grey, weathered, weak to medium strong, close to moderately close bedding, fair quality, occ. limestone layers (10 to 20mm thick).		4	SPT	100+									4 8 50 38 (88)
			5	SPT	100+									
			6	SPT	100+									Water level measured @ 4.3m @ completion.
			7	SPT	100+									
220.35 6.25	End of borehole.		8	SPT	100+									

JOE MTO 07-6-JEG1.GPJ ONTARIO.MOT.GDT 12/17/08

+ 3, × 3: Numbers refer to
Sensitivity

○ 150 UNCONFINED SHEAR STRENGTH INFERRED FROM POCKET PENETROMETER READINGS

RECORD OF BOREHOLE No 06A-2

1 OF 1

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			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	PENETR. RESISTANCE STANDARD ● DYN. CONE			PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa								WATER CONTENT (%)		
								○ UNCONFINED	● QUICK TRIAXIAL	+ FIELD VANE						× LAB VANE		
231.50 0.00	Ground							20	40	60	80	100	10	20	30	GR SA SI CL		
230.74 0.76	FILL Brown, moist, compact, consisting mainly of sand and gravel, 40mm stones.						231									41 38 15 6 (22)		
	100 mm ASPHALT		1	SPT	23													
	FILL Brown, moist, compact to dense, consisting of silty sand amnd gravel.		2	SPT	48		230											
229.52 1.98																		
	FILL Brown, moist to wet, loose to compact, consisting of silty clay with embedded sand and gravel.		3	SPT	8		229											
			4	SPT	7		228											
			5	SPT	10													
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.		6	SPT	30		227									6 8 55 31 (87)		
226.01 5.49			7	SPT	100+		226											
	SHALE BEDROCK Grey, slightly weathered, weak, close to moderately close bedding, fair quality, occ. limestone layers (10 to 20mm thick).		8	SPT	100+		225									8 16 42 33 (75)		
224.64 6.86			9	SPT	100+		224											
	SHALE BEDROCK Grey, unweathered, medium strong, close to moderately close bedding, fair quality, occ. limestone layers (10 to 20mm thick).		10	SPT	100+													
			11	SPT	100+		223											
222.20 9.30	End of borehole.		12	SPT	100+											Borehole dry and open @ completion.		

JOE MTO 07-6-JEG1.GPJ ONTARIO.MOT.GDT 12/18/08

+ 3, X 3: Numbers refer to
Sensitivity

○ 150 UNCONFINED SHEAR STRENGTH INFERRED FROM POCKET PENETROMETER READINGS

RECORD OF BOREHOLE No 06A-3

1 OF 2

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

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	PENETR. RESISTANCE STANDARD ● DYN. CONE		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							WATER CONTENT (%)
								○ UNCONFINED	+ FIELD VANE						
						● QUICK TRIAXIAL	× LAB VANE								
231.08	Ground						20	40	60	80	100	10	20	30	GR SA SI CL
0.00	50 mm TOPSOIL.		1	SPT	22										
	Silty CLAY TILL (CI) Brown, moist, very stiff to hard, embedded sand and gravel.		2	SPT	100+										
230.17															
0.91															
			RC3	NQ											Recovery - 80%, RQD - 100%
	SHALE BEDROCK Grey, weathered, weak to medium strong, close to moderately close bedding, fair quality, occ. limestone layers (10 to 20mm thick).		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															
			RC7	NQ											Recovery - 100%, RQD - 80%
	SHALE BEDROCK Grey, unweathered, medium strong, excellent quality.		RC8	NQ											Recovery - 100%, RQD - 95%
			RC9	NQ											Recovery - 100%, RQD -

Continued Next Page

+ 3, × 3: Numbers refer to
Sensitivity

○ 150 UNCONFINED SHEAR STRENGTH INFERRED FROM POCKET PENETROMETER READINGS

METRIC

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	PENETR. RESISTANCE STANDARD ● DYN. CONE					PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa										WATER CONTENT (%)		
								○ UNCONFINED + FIELD VANE										W _p W W _L		
								● QUICK TRIAXIAL × LAB VANE												
							20	40	60	80	100									
	SHALE BEDROCK Grey, unweathered, medium strong, excellent quality. (continued)						221										95%			
								220										Recovery - 100%, RQD - 90%		
			RC10	NQ				219										Recovery - 100%, RQD - 95%		
								218										Uniaxial Compressive Strength = 29.8 MPa		
			RC11	NQ				217										Recovery - 100%, RQD - 95%		
								216										Uniaxial Compressive Strength = 36.0 MPa		
			RC12	NQ				215										Recovery - 100%, RQD - 92%		
								214										Recovery - 100%, RQD - 90%		

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.	

Coarse

Grain Size Distribution Data

Sieve Size (Imperial)	Sieve Size (Metric)	Percent Passing
3/8"	9.5mm	5
1/2"	12.5mm	8
3/4"	16mm	10
1"	25mm	12
1 1/2"	37.5mm	15
2"	50mm	17
2 1/2"	62.5mm	22
3"	75mm	23
3 1/2"	87.5mm	27
4"	100mm	31
4 1/2"	112.5mm	37
5"	125mm	45
5 1/2"	137.5mm	58
6"	150mm	74
6 1/2"	162.5mm	85
7"	175mm	94
7 1/2"	187.5mm	99

LEGEND

BH	SAMPLE	SYMBOL
06A-2	1.52	●

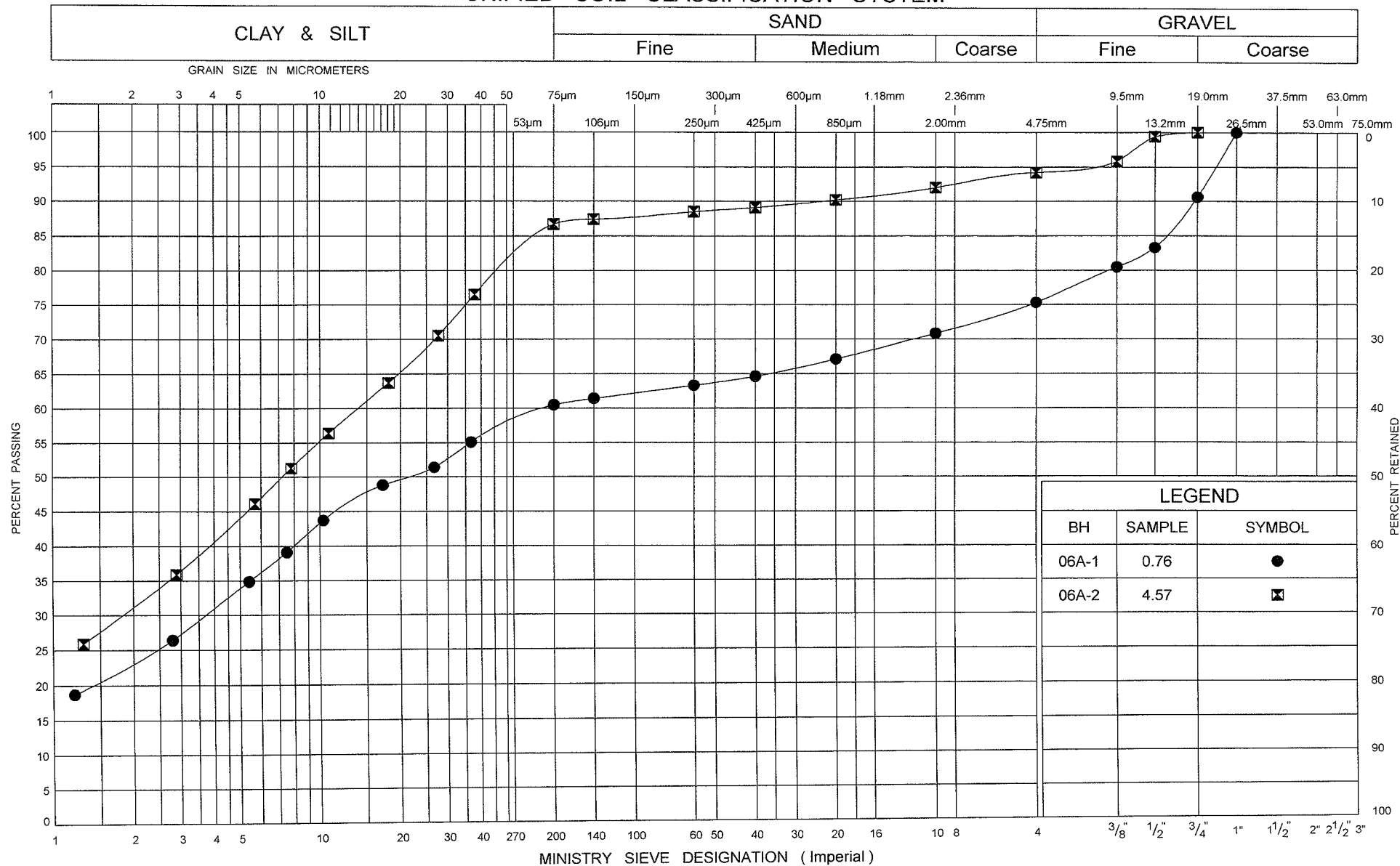
BH	SAMPLE	SYMBOL
06A-2	1.52	●
$\frac{3}{8}$ "	$\frac{1}{2}$ "	$\frac{3}{4}$ " 1" $1\frac{1}{2}$ " 2" $2\frac{1}{2}$ "

Ministry of
Transportation

FILL

HWY 26, Thornbuy to Meaford

UNIFIED SOIL CLASSIFICATION SYSTEM



GRAIN SIZE DISTRIBUTION

SILTY CLAY TILL

FIG No 2

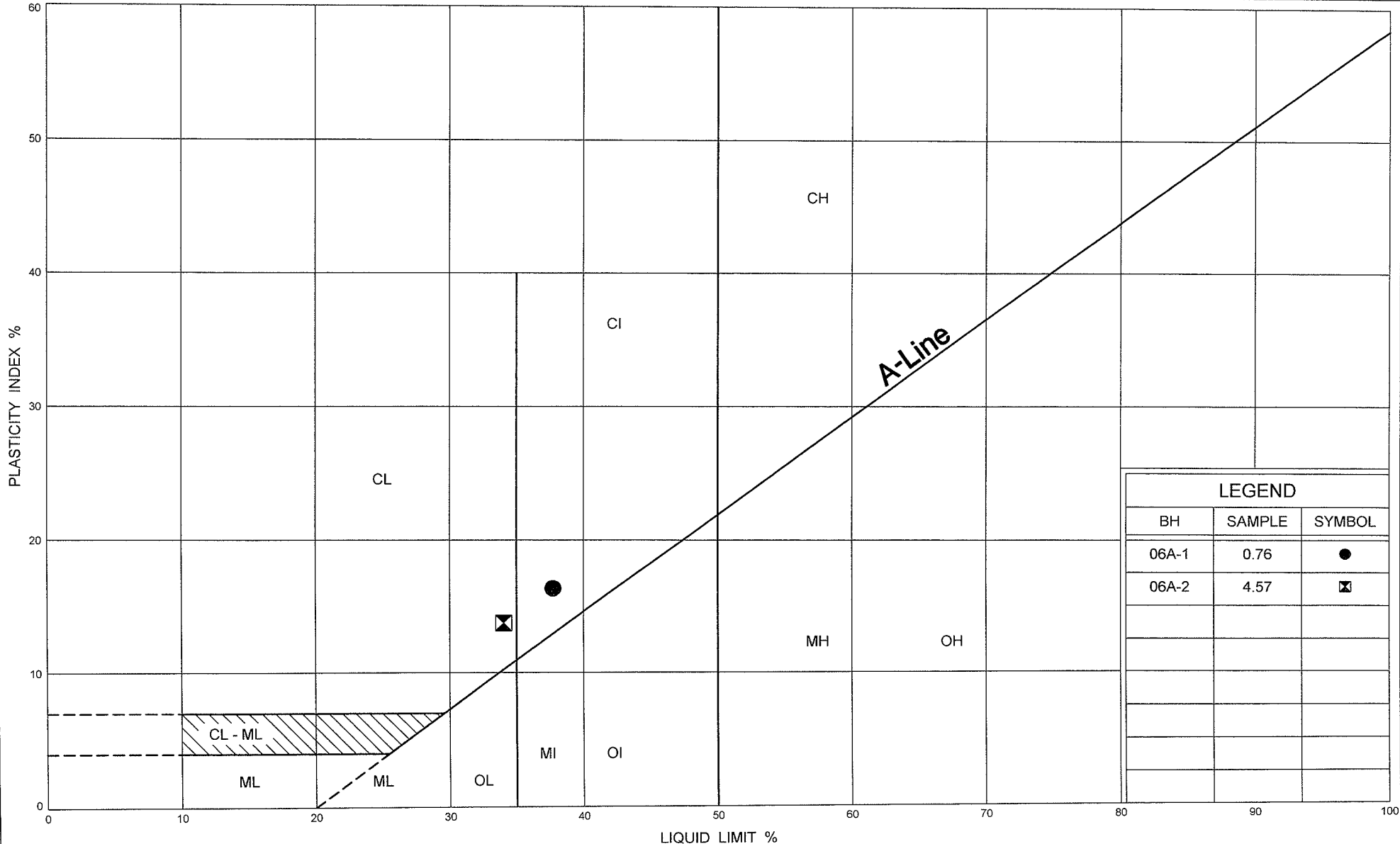
GWP 57-00-00

HWY 26, Thornbury to Meaford



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

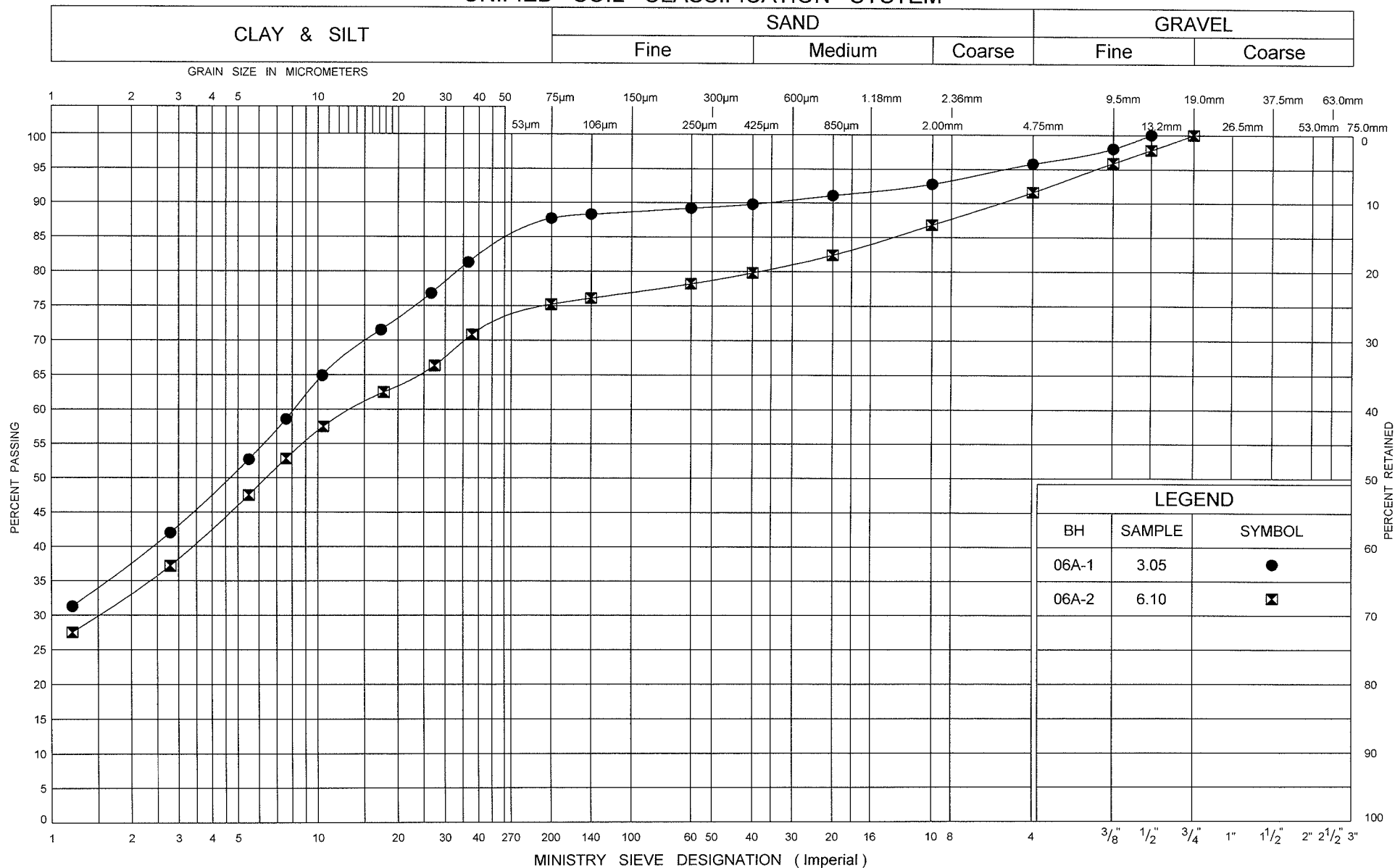
ONTARIO MOT PLASTICITY CHART 07-6-IEG1.GPJ ONTARIO MOT.GDT 01/24/08



PLASTICITY CHART SILTY CLAY TILL

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

UNIFIED SOIL CLASSIFICATION SYSTEM



GRAIN SIZE DISTRIBUTION

WEATHERED SHALE

FIG No 4

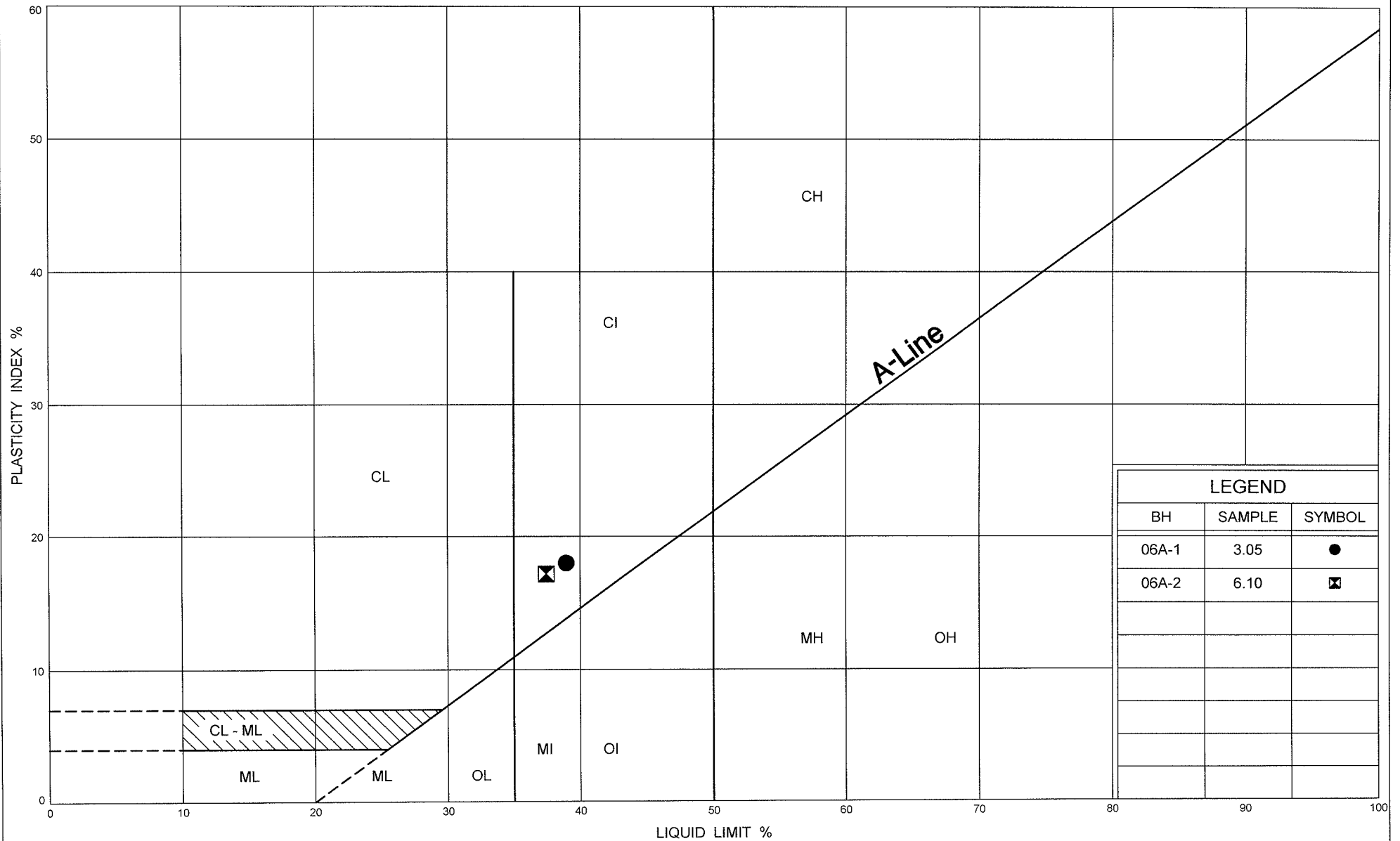
GWP 57-00-00

HWY 26, Thornbury to Meaford



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PLASTICITY CHART WEATHERED SHALE

FIG No 5

GWP 57-00-00

HWY 26, Thornbury to Meaford



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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

NOTES/COMMENTS:

Trow Associates Inc.

Author initials/C:\Documents and Settings\umecoal\My Documents\Fax Template.doc

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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

Date

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

07-6-IEG1-8-469C
Final Report
Appendix C
December 17, 2008

Appendix C

Limitations of Report

APPENDIX C

LIMITATIONS OF REPORT

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

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

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

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

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

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

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

07-6-IEG1-8-469C
Final Report
Appendix D
December 17, 2008

Appendix D
Site Photographs



Station 25+314 – Downstream end (Meaford Creek)



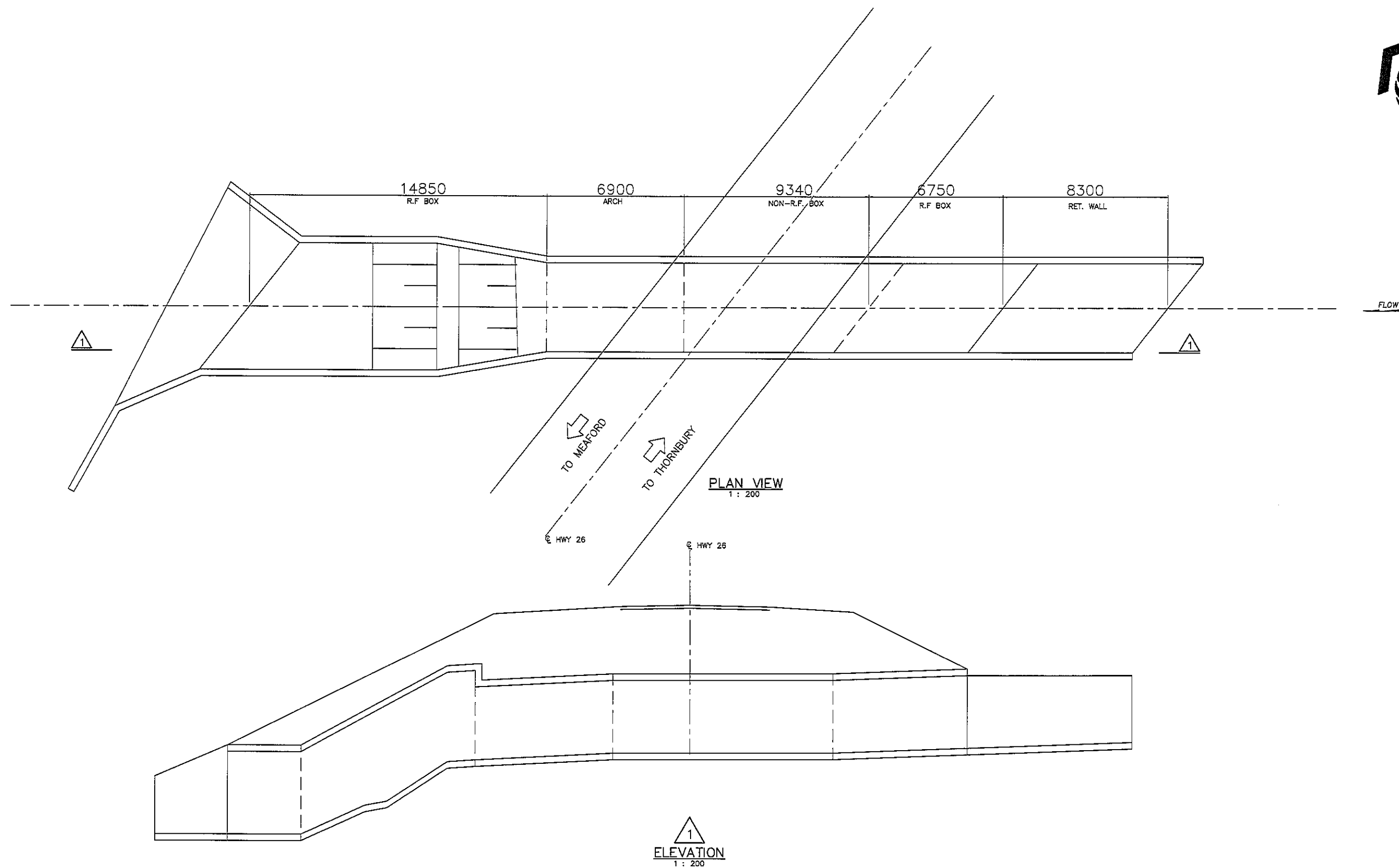
Station 25+314 – Looking upstream (south)



Station 25+314 – Downstream end (Meaford Creek)



Station 25+314 – Upstream end (south)



HIGHWAY 26 THORN BURY TO MEAFORD
W.P. 57-00-00

MEAFORD CREEK CULVERT
SITE 8-469C



CULVERT INSPECTION
AND
EVALUATION REPORT

FIGURE
1

Highway 26
W.P. 57-00-00

Site No. 8-469C

June 1, 2005



Photo 1: General view of road, looking west.

Highway 26
W.P. 57-00-00

Site No. 8-469C

June 1, 2005



Photo 3: North elevation.



Photo 2: South elevation.



Photo 4: Looking downstream.

Highway 26
W.P. 57-00-00

Site No. 8-469C

June 1, 2005



Photo 5: Looking south through culvert.

Highway 26
W.P. 57-00-00

Site No. 8-469C

June 1, 2005



Photo 7: Undermining at outlet.



Photo 6: Looking north through culvert.



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).



Photo 11: View of arch, looking north.



Photo 12: View of arch, looking south.

Highway 26
W.P. 57-00-00

Site No. 8-469C

June 1, 2005



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



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

Highway 26
W.P. 57-00-00

Site No. 8-469C

June 1, 2005



Photo 15:
Floor slab failure at arch,
looking north.



Photo 16:
View of arch soffit, looking
north.