December 2015

FOUNDATION INVESTIGATION AND DESIGN REPORT

Culvert Rehabilitation, Tedford Drain
Site No. 13-403/C, Station 19+290
Highway 401/Highway 40 Interchange Reconfiguration
Chatham-Kent, Ontario, GWP 3093-09-00
Ministry of Transportation, West Region

Submitted to:

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REPORT

Report Number: 13-1132-0111-1000-R11

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FOUNDATION INVESTIGATION REPORT

CULVERT REHABILITATION, TEDFORD DRAIN
SITE NO. 13-403/C, STATION 19+290
HIGHWAY 401/HIGHWAY 40 INTERCHANGE RECONFIGURATION
CHATHAM-KENT, ONTARIO
GWP 3093-09-00
MINISTRY OF TRANSPORTATION - WEST REGION

1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by Dillon Consulting Limited (Dillon) on behalf of the Ministry of Transportation, Ontario (MTO) to carry out foundation investigations as part of the detailed design work for GWP 3093-09-00. The project involves the detailed design for the reconfiguration of the Highway 401 and Highway 40 (Communication Road) interchange as well as the realignment of Pinehurst Line and reconstruction of the Highway 401 eastbound lanes. This report addresses the proposed rehabilitation of the culvert at Tedford Drain (Site 13-403/C) at Station 19+290 on Highway 401, east of Chatham, Ontario in the Municipality of Chatham-Kent.

The purpose of the foundation investigation is to explore the subsurface conditions at the location of the proposed culvert rehabilitation by drilling boreholes and carrying out in situ testing and laboratory testing on selected samples. The terms of reference for the scope of work are outlined in the MTO's Request for Proposal (RFP) and in Golder's change order 13-1132-0111-1000-C01 dated April 16, 2015. The work was carried out in accordance with our Quality Control Plan for Foundations Engineering dated February 27, 2014.

Dillon provided Golder with preliminary drawings for this project in digital format. In addition, the Preliminary Design Report (PDR) and Design Build Ready Report (DBRR) package were provided by the MTO.

2.0 SITE DESCRIPTION

The subject culvert is situated at Station 19+290 on Highway 401, approximately 650 metres east of Harwich Road in the Geographic Township of Harwich in Municipality of Chatham-Kent, Ontario. The Municipal Centre of Chatham is approximately 12 kilometres west of the site. The location of the culvert is shown on the Key Plan, Figure 1.

This section of Highway 401 is currently a six lane divided highway with paved shoulders. It is generally oriented east-west in the vicinity of the subject site. Tedford Drain flows in the culvert from south to north beneath Highway 401. The existing culvert is a concrete rigid frame open footing (RFO) structure with the following characteristics:

Dimensions (m)	Obvert Elevation (m)		Construction
	Lt[1]	Rt[1]	
4.42 x 2.59 x 59.80	187.0	187.0	RFO

NOTE: 1. When facing the direction of increasing chainage, Lt and Rt are defined as Left and Right of centreline, respectively.

The banks of the Tedford Drain channel immediately upstream and downstream of the culvert are grass covered with concrete retaining walls on all four corners. The channel is concrete lined and trapezoidal in shape throughout the culvert and just beyond each end. Tedford Drain flows through fields adjacent to Highway 401. Site photographs are provided in Appendix B.

The culvert is situated in a rural area with low relief. Ground surface elevations in the vicinity of the culvert range from about 186.5 to 189 metres.

2.1 Site Geology

The project area is located within the Bothwell Sand Plains physiographic region. This region is characterized by sands spread thinly over the clay floor.[1] The overburden in the area of the site generally consists of glaciolacustrine deposits of silty clay to silt overlain by silty sand and sand.[2]

The geological mapping indicates that the underlying bedrock consists of black bituminous shale of the Kettle Point Formation of the Port Lambton Group, upper Devonian age.[3] The bedrock surface at the site is at about elevation 160 metres, with the overburden thickness being about 26 to 29 metres.[4]

[1] Chapman, L.J., and Putnam, D.F., 1984: Physiography of Southern Ontario; Ontario Geological Survey, Special Volume 2, 270p. Accompanied by Map. P.2715 (coloured), scale 1:600,000.

[2] Kelly, R.I. 1991: Quaternary Geology of the Chatham - Wheatley Area. Southern Ontario; Ontario Geological Survey. Map 163, scale 1:50,000.

[3] Sanford B.V., 1969: Geology Toronto-Windsor Area, Ontario; Ontario Geological Survey of Canada Map 1263A, Scale 1:250,000.

[4] Sado, E.V. and Faught, R.B. 1981: Bedrock Topography of the Chatham Area, Southern Ontario. Ontario Geological Survey Preliminary Map No.P.2436, Scale 1:50,000.

3.0 INVESTIGATION PROCEDURES

The field work for the investigation was carried out on July 23, 2015, during which time 2 boreholes were drilled at the locations shown on the Borehole Location Plan, Drawing 1.

The boreholes were drilled using track-mounted CME 75 drilling equipment supplied and operated by a specialist drilling contractor. Samples of the overburden were typically obtained at depth intervals of 0.75 metres using 50 millimetre outside diameter split spoon sampling equipment in accordance with the Standard Penetration Test (SPT) procedures (ASTM D1586).

The recorded SPT N values are noted on the Record of Borehole sheets. The SPT resistance, or N value, is defined as the number of blows required by a 63.5 kilogram hammer dropped from a height of 760 millimetres to drive a split-spoon sampler a distance of 300 millimetres after an initial 150 millimetres of penetration. The results of the SPT testing as presented on the Record of Borehole sheets, Drawing 1 and in Section 4.0 of this report are unmodified (not standardized for hammer efficiency, borehole diameter, rod length, etc.). The samplers used in the investigation limit the maximum particle size that can be sampled and tested to about 40 millimetres. Therefore, particles or objects that may exist within the soils that are larger than this dimension will not be sampled or represented in the grain size distributions. Larger particle sizes, including cobbles and boulders, are known to be present in the silty clay till and clayey silt till as discussed in the text of this report.

Groundwater conditions in the boreholes were observed throughout the drilling operations. The boreholes were backfilled in accordance with current MTO procedures and Ontario Regulation 903 (as amended).

The field work was monitored on a full-time basis by an experienced member of our staff who located the boreholes in the field, monitored the drilling, sampling and in situ testing operations and logged the boreholes. The samples were identified in the field, placed in labelled containers and transported to our London laboratory for further examination and testing. Index and classification tests, consisting of water content determinations, Atterberg limits, and grain size distribution analyses, were carried out on selected samples. The results of the testing are shown on the Record of Borehole sheets and in Appendix A.

The as-drilled borehole locations and ground surface elevations are shown on the Record of Borehole sheets and on Drawing 1. The table below summarizes the coordinates, ground surface elevations and depths of the boreholes.

Borehole	Location (m)		Ground Surface Elevation	Borehole Depth
	Northing	Easting	(m)	(m)
701	4 695 342	343 037	186.52	6.55
702	4 695 357	343 062	186.85	6.55

4.0 SUBSURFACE CONDITIONS

4.1 Site Stratigraphy

The detailed subsurface soil and groundwater conditions encountered in the boreholes, together with the results of the in situ testing and the laboratory testing carried out on selected samples, are given on the attached Record of Borehole sheets following the text of this report and in Appendix A. The stratigraphic boundaries shown on the Record of Borehole sheets are inferred from non-continuous samples and observations of drilling resistance and, therefore, may represent transitions between soil types rather than exact planes of geological change. Further, the subsurface conditions will vary between and beyond the borehole locations.

The boreholes drilled at the site generally encountered the existing surficial topsoil overlying silty clay till and clayey silt till. Although not specifically encountered in the boreholes, fill materials associated with the existing retaining walls on the south end may also be present.

The locations and elevations of the boreholes, together with the interpreted stratigraphic profile, are shown on Drawing 1. A detailed description of the subsurface conditions encountered in the boreholes is provided on the Record of Borehole sheets and is summarized in the following sections.

4.2 Soil Conditions

4.2.1 Topsoil

Topsoil was encountered at the ground surface in each borehole. The topsoil was 150 and 200 millimetres thick. Materials designated as topsoil in this report were classified solely based on visual and textural evidence. Testing of organic content or for other nutrients was not carried out. Therefore, the use of materials classified as topsoil cannot be relied upon for support and growth of landscaping vegetation.

4.2.2 Silty Clay Glacial Till

Silty clay glacial till was encountered beneath the topsoil in boreholes 701 and 702 from elevations 186.3 and 186.7 metres, respectively. The very stiff silty clay till was 2.9 metres thick with measured N values ranging from 15 to 22 blows per 0.3 metres. Water contents of the samples ranged from 18 to 20 per cent. Cobbles and boulders should be expected in the silty clay till.

The silty clay till is of intermediate plasticity based on the Atterberg limits determinations carried out on a sample obtained during standard penetration testing. The plastic limit was 25 per cent, the liquid limit was 40 per cent, and the plasticity index was 15 per cent. The Atterberg limits data for the silty clay till are presented on Figure A-3. A grain size distribution curve for a sample of the silty clay till is provided on Figure A-1.

4.2.3 Clayey Silt Glacial Till

Layers of stiff clayey silt glacial till were encountered beneath the silty clay till in boreholes 701 and 702 from elevations 183.5 and 183.8 metres, respectively. Both boreholes were terminated in the clayey silt till after exploring the layer for 3.5 metres. Measured N values for the clayey silt till layers ranged from 8 to 13 blows per 0.3 metres. Water contents of the samples ranged from 12 to 24 per cent. Cobbles and boulders should be expected in the clayey silt till.

The clayey silt till is of low plasticity based on the Atterberg limits determinations carried out on samples obtained during standard penetration testing. The plastic limit ranged from 14 to 16 per cent, the liquid limit from 23 to 27 per cent, and the plasticity index from 7 to 11 per cent. The Atterberg limits data for the clayey silt till are presented on Figure A-3. Grain size distribution curves for samples of the clayey silt till are provided on Figure A-2.

4.3 Groundwater Conditions

Groundwater conditions were observed during and on completion of drilling. The groundwater level in boreholes 701 and 702 was not established at the completion of drilling. A summary of the groundwater levels is provided in the table below.

Borehole	Ground Surface Elevation (m)	Encountered Groundwater Elevation (m)
701	186.52	*
702	186.85	*

*Groundwater not established

The corresponding water level in the watercourse was measured at elevation 185.0 metres on July 23, 2015.

Based on the surrounding topography, the soil colour change from brown to grey and water level in the drain, the groundwater level is inferred to typically be at about elevation 185.0 metres. The groundwater levels are expected to fluctuate seasonally and are expected to be higher during periods of sustained precipitation or during spring snow melt conditions.

5.0 MISCELLANEOUS

The investigation was carried out using equipment supplied and operated by London Soil Test Inc., an Ontario Ministry of Environment licensed well contractor. The field operations were supervised by Mr. Brett Thorner, E.I.T. under the direction of the Field Investigation Manager, Mr. David J. Mitchell. The laboratory testing was carried out at Golder Associates' London laboratory under the direction of Mr. Chris M. Sewell. The laboratory is an accredited participant in the MTO Soil and Aggregate Proficiency Program and is certified by the Canadian Council of Independent Laboratories for testing Types C and D aggregates. This report was prepared by Mr. Brett Thorner, E.I.T. under the direction of the Project Engineer Ms. Dirka U. Prout, P.Eng and Team Leader, Dr. Storer J. Boone, P.Eng. This report was reviewed by Mr. W. Michael Kellestine, P.Eng., a Senior Consultant with Golder Associates. Mr. Fintan J. Heffernan, P.Eng., the Designated MTO Contact and Quality Control Auditor for this assignment conducted an independent quality review of the report.

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PART B

FOUNDATION DESIGN REPORT

CULVERT REHABILITATION, TEDFORD DRAIN
SITE NO. 13-403/C, STATION 19+290
HIGHWAY 401/HIGHWAY 40 INTERCHANGE RECONFIGURATION
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MINISTRY OF TRANSPORTATION - WEST REGION

6.0 ENGINEERING RECOMMENDATIONS

6.1 General

This section of the report provides recommendations on the foundation aspects of the design of proposed retaining walls to be carried out as part of the minor rehabilitation of Culvert Site 13-403/C at Station 19+290 on Highway 401 in the Geographic Township of Harwich in Municipality of Chatham-Kent, Ontario.

The recommendations are based on interpretation of the factual data obtained from the boreholes advanced during the investigation at this site. The interpretation and recommendations are intended to provide the designers with sufficient information to design the proposed foundations. As such, where comments are made on construction, they are provided only in order 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 methods, and scheduling.

The existing culvert is a 59.8 metre long concrete RFO structure with a 4.4 metre span and a 2.6 metre high opening with invert elevations of 184.4 metres at the inlet and outlet. The existing concrete retaining walls have a maximum height of about 3.0 metres. Based on information provided by Dillon, it is understood that the existing retaining walls on the south (inlet) side will be replaced as part of the minor rehabilitation of the culvert. It has been indicated by Dillon that consideration is being given to constructing the retaining walls as reinforced concrete gravity and cantilever, gabion or reinforced soil system (RSS) walls. The retaining wall options are discussed below.

6.2 Retaining Walls

6.2.1 Retaining Wall Options

Reinforced Concrete Gravity and Cantilever Walls

Construction of reinforced concrete gravity or cantilever walls is geotechnically feasible. Compared to gabion walls or RSS walls, footings for CIP gravity and cantilever walls must be constructed with a frost cover of 1.2 metres. This may result in a longer foundation construction time compared other wall types which do not require a similar embedment depth.

Gabion Walls

Gabion walls do not require an embedment depth equivalent to the frost depth provided they are founded on a granular pad of 300 millimetres compacted thickness, and the foundations have adequate embedment to provide a stable structure. Advantages of gabion walls compared to more rigid structures include the ability to accommodate differential settlements and dissipation of the energy of flowing water. They are also free-draining provided an adequate filter is placed behind the wall. Gabion walls can be constructed relatively quickly with minimal equipment and materials.

RSS Walls

The height of the retaining walls will be relatively low. Therefore, a reinforced soil system wall utilizing an interlocking block system and geogrid reinforcement is a geotechnically feasible alternative. RSS walls are proprietary systems which are to be designed by the supplier and constructed in accordance with their specifications. The internal stability of the mechanically-reinforced soil walls should be verified by the RSS supplier/designer. If an RSS block wall system is selected, the global stability of the wall system including bearing eccentricity and sliding resistance should be reviewed by the MTO designer prior to construction. Depending on the design approach selected, an embedment depth equivalent to the frost depth may not be required for foundations of an RSS block system wall. This wall type can be constructed relatively quickly and inexpensively using small equipment.

6.2.2 Foundations – Retaining Walls

Reinforced concrete gravity and cantilever walls founded on concrete strip footings must be provided with a frost cover of 1.2 metres below the adjacent ground or thermal equivalent. Assuming the ground surface at the new retaining wall location is near elevation 186.5 metres, foundations for these wall types must be founded at or below elevation 185.3 metres in the stiff to very stiff glacial till.

Gabion walls may be founded directly on a 300 millimetre thick compacted Granular A pad. Non-woven geotextile is to be placed between the gabions and the backfill. The gabion wall is to be constructed in accordance with Ontario Provincial Standard Specifications (OPSS) 1430 and 1860, and the manufacturer's specifications.

RSS walls may be designed such that the facing blocks are built on a levelling pad constructed with Granular A to a minimum thickness of 300 millimetres. Depending on the design selected by the RSS supplier, it may not be necessary to provide 1.2 metres of earth cover or thermal equivalent for frost protection. However the foundations must have adequate embedment to provide a stable structure. Typically the embedment depth, defined as the distance between the top of the levelling pad and the top of the adjoining finished grade, is a minimum of 500 millimetres.

All retaining wall foundations must be protected against scour as noted in the Canadian Highway Bridge Design Code (CHBDC) Section 1.9.5.

If required, engineered fill used to backfill subexcavated areas should consist of OPSS Granular B Type II or III and should be compacted to at least 95 per cent standard Proctor maximum dry density.

Retaining walls founded on the native silty clay till may be designed using a factored geotechnical resistance at Ultimate Limit States (ULS) of 225 kilopascals and a geotechnical resistance at Serviceability Limit States (SLS) of 150 kilopascals. The SLS values correspond to 25 millimetres of settlement.

6.2.3 Resistance to Lateral Forces

The resistance to lateral forces/sliding resistance between the retaining walls and the subgrade soils should be calculated in accordance with Section 6.7.5 of the CHBDC. Each retaining wall shall be checked for overturning. Assuming that the founding soils are not loosened/disturbed during excavation and footing construction, the following angles of friction and corresponding unfactored coefficient of friction, tan δ, may be used for the interaction between the base of the wall and the founding soil:

Wall Type	Interaction	Angle of Interface Friction, δ (degrees)	Coefficient of Interface Friction, Tan δ
Reinforced Concrete Gravity or Cantilever Wall on cast-in-place concrete strip footings	Cast-in-place concrete footing on glacial till	30	0.58
Gabion Wall	Gabion basket on Granular A leveling pad	30	0.58
RSS Block System Wall	Pre-cast concrete block facing units on Granular A levelling pad	30	0.58

6.2.4 Lateral Earth Pressures for Design

The lateral pressures acting on the proposed retaining walls will depend on the type and method of placement of the backfill materials, on the nature of the soil behind the backfill, on the magnitude of surcharge including construction loadings, on the freedom of lateral movement of the structure, and on the drainage conditions behind the walls.

The following recommendations are made concerning the design of the walls in accordance with the current CHBDC. It should be noted that these design recommendations and parameters assume level backfill and ground surface behind the walls. Where there is sloping ground behind the walls, the coefficient of lateral earth pressure must be adjusted to account for the slope.

- Select, free-draining granular fill meeting the specifications of OPSS Granular A or Granular B Type III but with less than 5 per cent passing the 0.075 millimetre sieve should be used as backfill behind the walls. The fill should be compacted in loose lifts not greater than 200 millimetres in thickness in accordance with OPSS.PROV 501. Longitudinal drains and weep holes should be installed to provide positive drainage of the granular backfill.

- A compaction surcharge equal to 12 kilopascals should be included in the lateral earth pressures for the structural design in accordance with CHBDC Figure 6.6. Compaction equipment should be used in accordance with OPSS.PROV 501.

- If the wall support does not allow lateral yielding, at-rest earth pressures should be assumed for geotechnical design. The granular fill should be placed in a zone with a width equal to at least 1.2 metres behind the walls (case (a) from commentary on CHBDC Figure C6.20).

- If the wall support allows lateral yielding (unrestrained structure, such as typically the case for retaining walls), active earth pressures may be used in the geotechnical design of the structure. The granular fill should be placed in a wedged shaped zone with a width equal to at least 1.2 metres at the footing level against a cut slope which begins at the footing level and extends upwards at a maximum inclination of 1 horizontal to 1 vertical (case (b) from commentary on CHBDC Figure C6.20).
- For walls backfilled using granular materials in accordance with case (b), the unrestrained case, the following parameters (unfactored) may be assumed:

	GRANULAR A	GRANULAR B TYPE III
Fill unit weight:	22 kN/m^3	21 kN/m^3
Coefficients of lateral earth pressure:		
'active' or unrestrained, K_a	0.27	0.31
‘passive’, K_p	3.7	3.3

6.3 Construction Considerations

6.3.1 General

Care should be taken during construction to avoid disturbance of the subgrade soils prior to constructing foundations for the replacement retaining walls. All existing fill and any topsoil, organics, and soft or loose soils should be stripped from the proposed founding areas prior to placement of base materials. Subgrade preparation should be performed and monitored in accordance with OPSS 902.

It is recommended that the footing excavations be carried out such that the final 0.5 metres of excavation is completed with a Quality Verification Engineer (QVE) experienced in geotechnical engineering on site. The prepared excavation bases should be inspected by the QVE and granular base materials should be placed immediately after inspection to protect the founding materials. If the granular base materials cannot be placed immediately after subgrade inspection (within 4 hours) a working mat of lean concrete should be used to protect the founding soils.

Sediment control such as silt fences and erosion control blankets may be required during construction.

6.4 Excavations and Groundwater Control

Excavations will extend through the existing surficial topsoil to the underlying native glacial till. The excavation for the retaining walls may likely extend below the inferred groundwater level of elevation 185.0 metres in order to provide adequate embedment, scour protection and frost protection. Seepage volumes from the glacial till are anticipated to be such that groundwater control may be achieved by using properly constructed and filtered sumps. Sumps should be maintained outside of the actual wall footing limits.

Surficial water seepage into the excavations should be expected and will be heavier during periods of sustained precipitation. Surface water runoff should be directed away from the excavations at all times. The existing culvert flows may need to be diverted/piped during construction.

Temporary open cut slopes within the fill materials should be maintained no steeper than 1 horizontal to 1 vertical and localized sloughing and ground movements should be expected. All excavations should be carried out in accordance with the latest edition of the Ontario Occupational Health and Safety Act and Regulations for Construction Projects. The native glacial till would be classified a Type 2 soil.

6.5 Staging and Temporary Roadway Protection

It is anticipated that the proposed works will not require excavation of the existing highway platform. However, should space be restricted and will not permit open cuts, a temporary support system should be installed to support the sides of the excavation and permit the use of vertical cuts. The temporary excavation support system should be designed and constructed in accordance with OPSS 539. The lateral movement of the temporary shoring system should meet Performance Level 2. The contractor is responsible for the complete detailed design of the protection system.

The support systems may be designed using the parameters provided in the table below. These parameters are provided to assist with design for the unfactored ultimate resistance and loading conditions and may not result in a temporary support design that adequately controls ground and structure displacements. Achieving adequate displacement control in accordance with the MTO performance criteria may require designs that result in a system that is stiffer than might otherwise be required based on the soil parameters provided in the table below.

Soil Type	Coefficient of Earth Pressure			Internal Angle of Friction (degrees)	Unit Weight (kN/m^3)
	Active, K_a	At Rest, K_o	Passive, K_p		
Glacial Till	0.31	0.47	3.3	32	22

The earth pressure coefficients identified above may be applied assuming a horizontal ground surface behind the retaining structure. Where the ground surface behind the retaining structure is sloped, the earth pressure coefficients provided in the table above must be increased.

7.0 MISCELLANEOUS

This section of the report was prepared by Mr. Brett Thorner, E.I.T. under the direction of the Project Engineer Ms. Dirka U. Prout, P.Eng. and the Team Leader, Dr. Storer J. Boone, P.Eng. The report was reviewed by Mr. W. Michael Kellestine, P.Eng., a Senior Consultant with Golder Associates. Mr. Fintan J. Heffernan, P.Eng., the Designated MTO Contact and Quality Control Auditor for this assignment, conducted an independent quality review of the report.

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LIST OF ABBREVIATIONS

The abbreviations commonly employed on Records of Boreholes, on figures and in the text of the report are as follows:

I. SAMPLE TYPE

AS	Auger sample
BS	Block sample
CS	Chunk sample
DS	Denison type sample
FS	Foil sample
RC	Rock core
SC	Soil core
SS	Split-spoon
ST	Slotted tube
TO	Thin-walled, open
TP	Thin-walled, piston
WS	Wash sample

II. PENETRATION RESISTANCE

Standard Penetration Resistance (SPT), N:
The number of blows by a 63.5 kg. (140 lb.) hammer dropped 760 mm (30 in.) required to drive a 50 mm (2 in.) drive open sampler for a distance of 300 mm (12 in.)

Dynamic Cone Penetration Resistance; N_d:
The number of blows by a 63.5 kg (140 lb.) hammer dropped 760 mm (30 in.) to drive uncased a 50 mm (2 in.) diameter, 60º cone attached to “A” size drill rods for a distance of 300 mm (12 in.).

PH: Sampler advanced by hydraulic pressure
PM: Sampler advanced by manual pressure
WH: Sampler advanced by static weight of hammer
WR: Sampler advanced by weight of sampler and rod

Piezo-Cone Penetration Test (CPT)
A electronic cone penetrometer with a 60° conical tip and a project end area of 10 cm^2 pushed through ground at a penetration rate of 2 cm/s. Measurements of tip resistance (Q_t), porewater pressure (PWP) and friction along a sleeve are recorded electronically at 25 mm penetration intervals.

III. SOIL DESCRIPTION

(a) Non-Cohesive (Cohesionless) Soils

Density Index Relative Density	**N Blows/300 mm or Blows/ft**
Very loose	0 to 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very dense	over 50

(b) Cohesive Soils
Consistency

	c_u, s_u kPa	c_u, s_u psf
Very soft	0 to 12	0 to 250
Soft	12 to 25	250 to 500
Firm	25 to 50	500 to 1,000
Stiff	50 to 100	1,000 to 2,000
Very stiff	100 to 200	2,000 to 4,000
Hard	over 200	over 4,000

IV. SOIL TESTS

w	water content
w_p	plastic limit
w_l	liquid limit
C	consolidation (oedometer) test
CHEM	chemical analysis (refer to text)
CID	consolidated isotropically drained triaxial test[1]
CIU	consolidated isotropically undrained triaxial test with porewater pressure measurement[1]
D_R	relative density (specific gravity, G_s)
DS	direct shear test
M	sieve analysis for particle size
MH	combined sieve and hydrometer (H) analysis
MPC	Modified Proctor compaction test
SPC	Standard Proctor compaction test
OC	organic content test
SO_4	concentration of water-soluble sulphates
UC	unconfined compression test
UU	unconsolidated undrained triaxial test
V	field vane (LV-laboratory vane test)
γ	unit weight

Note: 1 Tests which are anisotropically consolidated prior to shear are shown as CAD, CAU.

V. MINOR SOIL CONSTITUENTS

Per cent by Weight	**Modifier**	**Example**
0 to 5	Trace	Trace sand
5 to 12	Trace to Some (or Little)	Trace to some sand
12 to 20	Some	Some sand
20 to 30	(ey) or (y)	Sandy
over 30	And (non-cohesive (cohesionless)) or	Sand and Gravel
	With (cohesive)	Silty Clay with sand / Clayey Silt with sand

LIST OF SYMBOLS

Unless otherwise stated, the symbols employed in the report are as follows:

I. GENERAL

π	3.1416
ln x,	natural logarithm of x
$\log_{10}$	x or log x, logarithm of x to base 10
g	acceleration due to gravity
t	time
FoS	factor of safety

II. STRESS AND STRAIN

γ	shear strain
Δ	change in, e.g. in stress: $\Delta\,\sigma$
ε	linear strain
ε_v	volumetric strain
η	coefficient of viscosity
υ	Poisson's ratio
σ	total stress
σ'	effective stress ($\sigma' = \sigma - u$)
σ'_{vo}	initial effective overburden stress
$\sigma_1, \sigma_2, \sigma_3$	principal stress (major, intermediate, minor)
σ_{oct}	mean stress or octahedral stress $= (\sigma_1 + \sigma_2 + \sigma_3)/3$
τ	shear stress
u	porewater pressure
E	modulus of deformation
G	shear modulus of deformation
K	bulk modulus of compressibility

III. SOIL PROPERTIES

(a) Index Properties

$\rho(\gamma)$	bulk density (bulk unit weight)*
$\rho_d(\gamma_d)$	dry density (dry unit weight)
$\rho_w(\gamma_w)$	density (unit weight) of water
$\rho_s(\gamma_s)$	density (unit weight) of solid particles
γ'	unit weight of submerged soil ($\gamma' = \gamma - \gamma_w$)
D_R	relative density (specific gravity) of solid particles ($D_R = \rho_s / \rho_w$) (formerly G_s)
e	void ratio
n	porosity
S	degree of saturation

* Density symbol is ρ. Unit weight symbol is γ where $\gamma = \rho g$ (i.e. mass density multiplied by acceleration due to gravity)

(a) Index Properties (continued)

w	water content
w_l or LL	liquid limit
w_p or PL	plastic limit
I_p or PI	plasticity index = $(w_l - w_p)$
w_s	shrinkage limit
I_L	liquidity index = $(w - w_p) / I_p$
I_C	consistency index = $(w_l - w) / I_p$
e_{max}	void ratio in loosest state
e_{min}	void ratio in densest state
I_D	density index = $(e_{max} - e) / (e_{max} - e_{min})$ (formerly relative density)

(b) Hydraulic Properties

h	hydraulic head or potential
q	rate of flow
v	velocity of flow
i	hydraulic gradient
k	hydraulic conductivity (coefficient of permeability)
j	seepage force per unit volume

(c) Consolidation (one-dimensional)

C_c	compression index (normally consolidated range)
C_r	recompression index (over-consolidated range)
C_s	swelling index
C_α	secondary compression index
m_v	coefficient of volume change
c_v	coefficient of consolidation (vertical direction)
c_h	coefficient of consolidation (horizontal direction)
T_v	time factor (vertical direction)
U	degree of consolidation
σ'_p	pre-consolidation stress
OCR	over-consolidation ratio = σ'_p / σ'_{vo}

(d) Shear Strength

τ_p, τ_r	peak and residual shear strength
ϕ'	effective angle of internal friction
δ	angle of interface friction
μ	coefficient of friction = tan δ
c′	effective cohesion
c_u, s_u	undrained shear strength ($\phi = 0$ analysis)
p	mean total stress $(\sigma_1 + \sigma_3)/2$
p′	mean effective stress $(\sigma'_1 + \sigma'_3)/2$
q	$(\sigma_1 - \sigma_3)/2$ or $(\sigma'_1 - \sigma'_3)/2$
q_u	compressive strength $(\sigma_1 - \sigma_3)$
S_t	sensitivity

Notes:
1. $\tau = c' + \sigma' \tan \phi'$
2. shear strength = (compressive strength)/2

Golder Associates
London, Ontario

RECORD OF BOREHOLE No 701

METRIC

PROJECT 13-1132-0111
W.P. 3093-09-00
LOCATION N 4695342.2 , E 343036.5
ORIGINATED BY BT
DIST
HWY 40
BOREHOLE TYPE POWER AUGER, HOLLOW STEM
COMPILED BY LMK
DATUM GEODETIC
DATE July 23, 2015
CHECKED BY

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
186.52	GROUND SURFACE					
0.00	TOPSOIL, silty					
0.20	Brown SILTY CLAY TILL, some sand, trace gravel Very stiff Brown turning grey at about elev. 184.1m				186	
		1	SS	17		
		2	SS	22	185	1 12 43 44
		3	SS	15	184	
183.47 / 3.05	CLAYEY SILT TILL, some sand to sandy, trace gravel Stiff Grey					
		4	SS	11	183	
		5	SS	8		
		6	SS	8	182	3 24 49 24
		7	SS	10	181	
		8	SS	10		
179.97 / 6.55	END OF BOREHOLE Groundwater not established during drilling on July 23, 2015.				180	

Column headings: SOIL PROFILE; STRAT PLOT; SAMPLES; GROUND WATER CONDITIONS; ELEVATION SCALE; DYNAMIC CONE PENETRATION RESISTANCE PLOT 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; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

Water content plot annotation: 40

$+^3, \times^3$: Numbers refer to Sensitivity
$\circ^{3\%}$ STRAIN AT FAILURE

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London, Ontario

RECORD OF BOREHOLE No 702

METRIC

PROJECT 13-1132-0111

W.P. 3093-09-00 LOCATION N 4695357.3 , E 343062.2 ORIGINATED BY BT

DIST HWY 40 BOREHOLE TYPE POWER AUGER, HOLLOW STEM COMPILED BY LMK

DATUM GEODETIC DATE July 23, 2015 CHECKED BY

ELEV / DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
186.85	GROUND SURFACE						
0.00	TOPSOIL, silty						
0.15	Brown SILTY CLAY TILL, some sand, trace gravel Very stiff Brown turning grey at about elev. 184.7m						
		1	SS	16		186	
		2	SS	22		185	
		3	SS	18		184	
183.80 / 3.05	CLAYEY SILT TILL, some sand, with silty sand seams Stiff Grey	4	SS	13			4 21 51 24
		5	SS	9		183	
		6	SS	10		182	
						181	
		7	SS	8			7 31 38 24
180.30 / 6.55	END OF BOREHOLE Groundwater not established during drilling on July 23, 2015.						

DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100

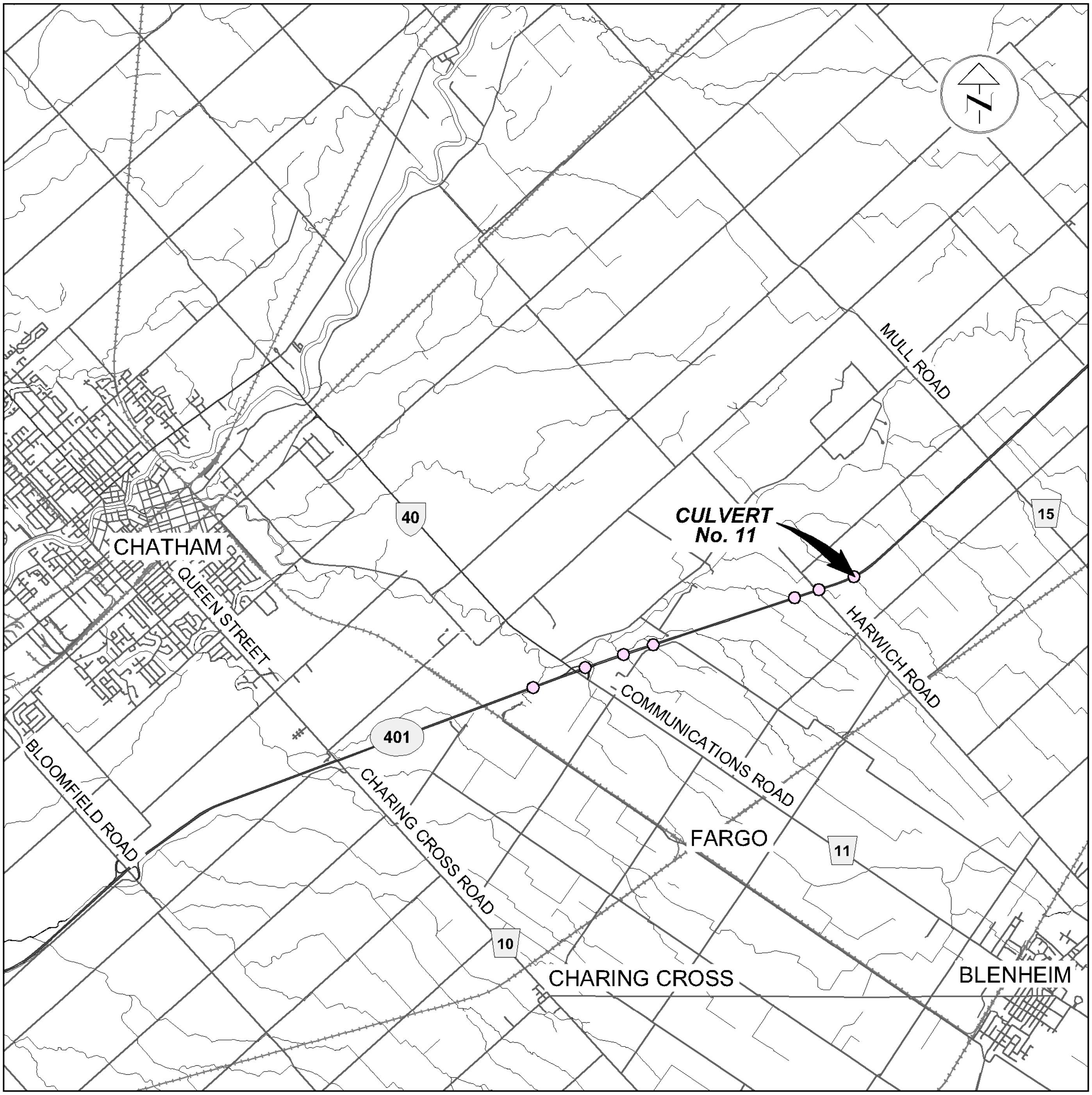
SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100

PLASTIC LIMIT W_P NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L

WATER CONTENT (%) 10 20 30

UNIT WEIGHT γ kN/m³

+³, ×³: Numbers refer to Sensitivity ○³% STRAIN AT FAILURE



REFERENCE

PLAN BASED ON CANMAP STREETFILES V.2008.

NOTE

THIS DRAWING IS TO BE READ IN CONJUNCTION WITH ACCOMPANYING TEXT.

PROJECT
CULVERT No. 11, TEDFORD DRAIN, SITE 13-403/C
HIGHWAY 401/40 INTERCHANGE RECONFIGURATION
GWP 3093-09-00

TITLE

KEY PLAN

Golder Associates
LONDON, ONTARIO

PROJECT No.		13-1132-0111	FILE No.	1311320111-1000-F11001
			SCALE AS SHOWN	REV. 0
CADD	WDF	Sept 23/15	**FIGURE 1**	
CHECK				

Drawing file: 1311320111-1000-F11001.DWG Nov 18, 2015 - 11:26am

METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES.

CONT No.
WP No. 3093–09–00

SHEET

CULVERT No. 11

HIGHWAY 401 / HIGHWAY 40 INTERCHANGE RECONFIGURATION

BOREHOLE LOCATIONS AND SOIL STRATA

Golder Associates

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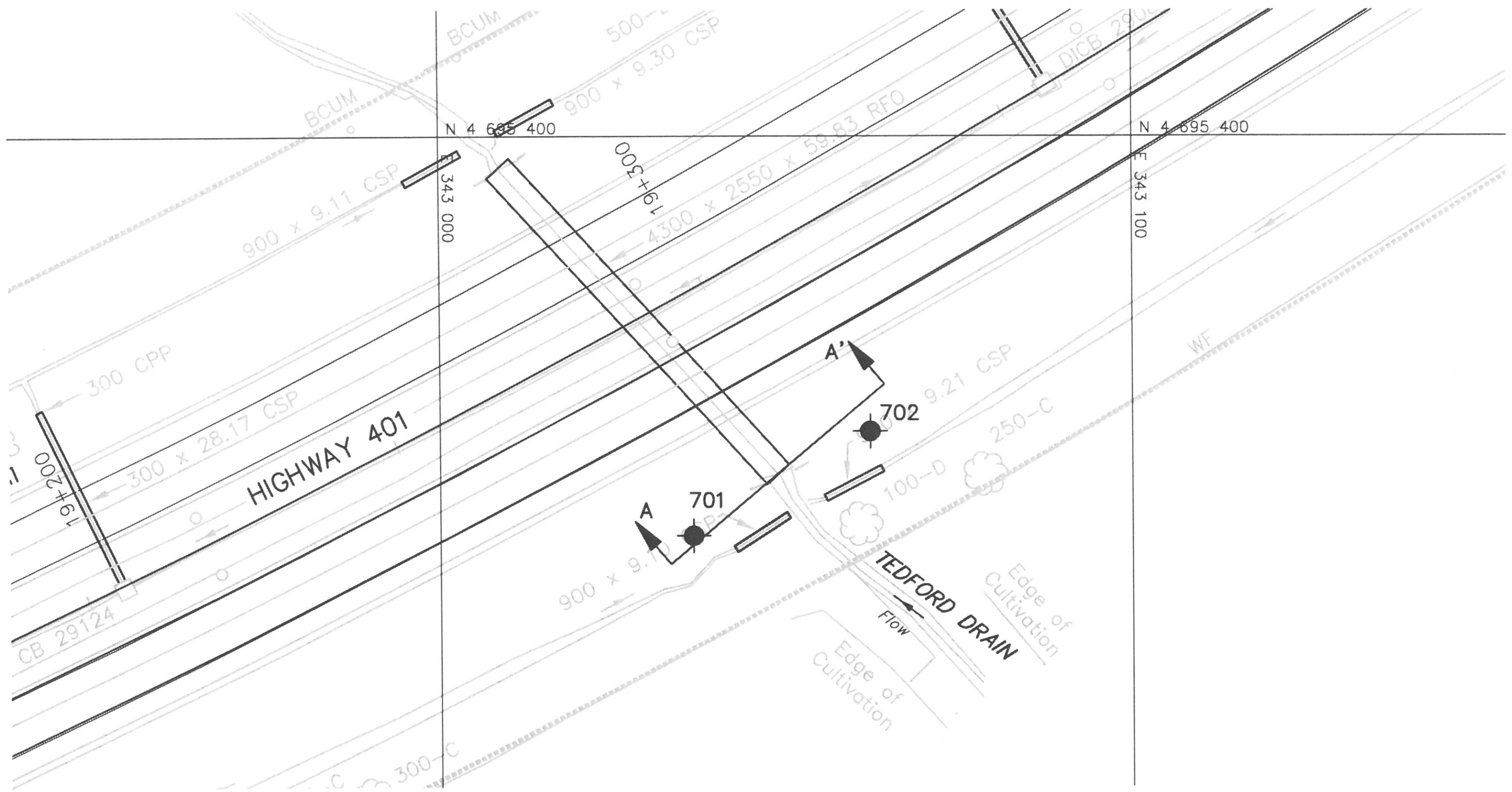
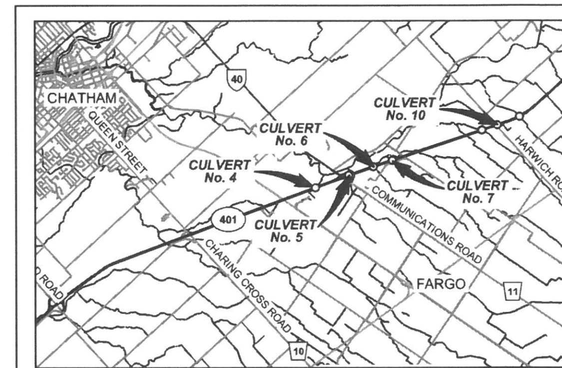


PLAN
SCALE
10 0 10 m

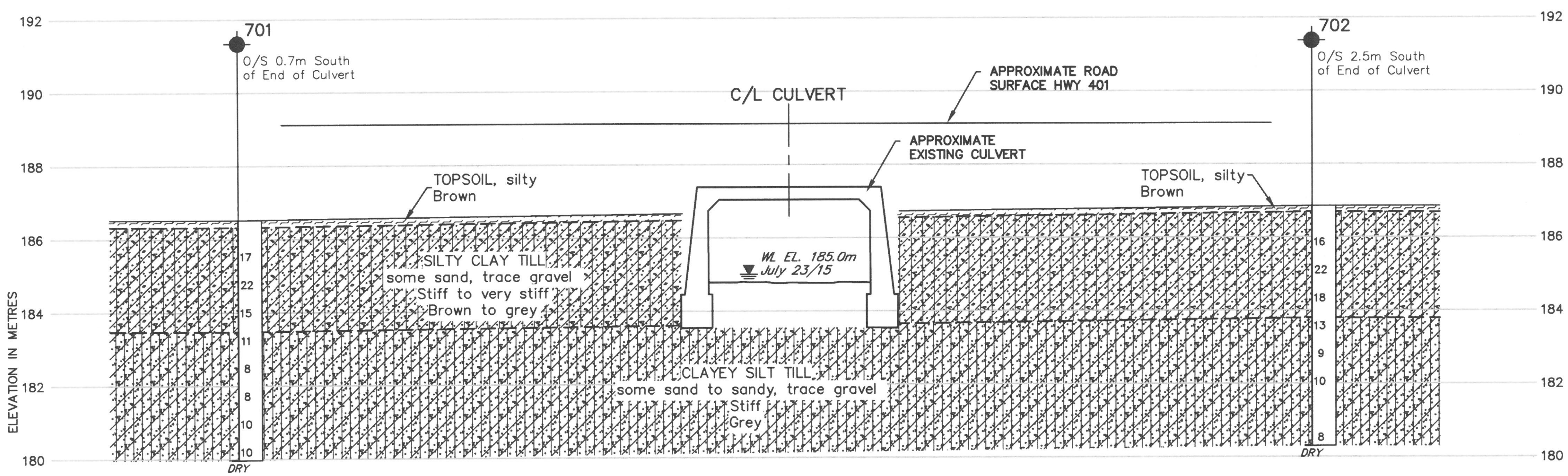


KEY PLAN
SCALE IN KILOMETRES
0 2 4

LEGEND

- Borehole – Current Investigation
- Seal
- Standpipe
- N — Standard Penetration Test Value
- 16 — Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- WL in Tedford Drain July 23, 2015.
- *DRY* — WL not established during drilling.

No.	ELEVATION	CO-ORDINATES (MTM ZONE 10) NORTHING	EASTING
701	186.52	4 695 342.2	343 036.5
702	186.85	4 695 357.3	343 062.2



SECTION A–A'
HORIZONTAL SCALE 1.5 0 1.5 m
VERTICAL SCALE 1.5 0 1.5 m

NOTES

This drawing is for subsurface information only. Surface details and features are for conceptual illustration.

The boundaries between soil strata have been established only at borehole locations. Between boreholes the boundaries are assumed from geological evidence.

REFERENCE

Base plans provided in digital format by Dillon
Received Aug. 18, 2015.

LICENSED PROFESSIONAL ENGINEER D. B. PROUT Dec 16/15 PROVINCE OF ONTARIO

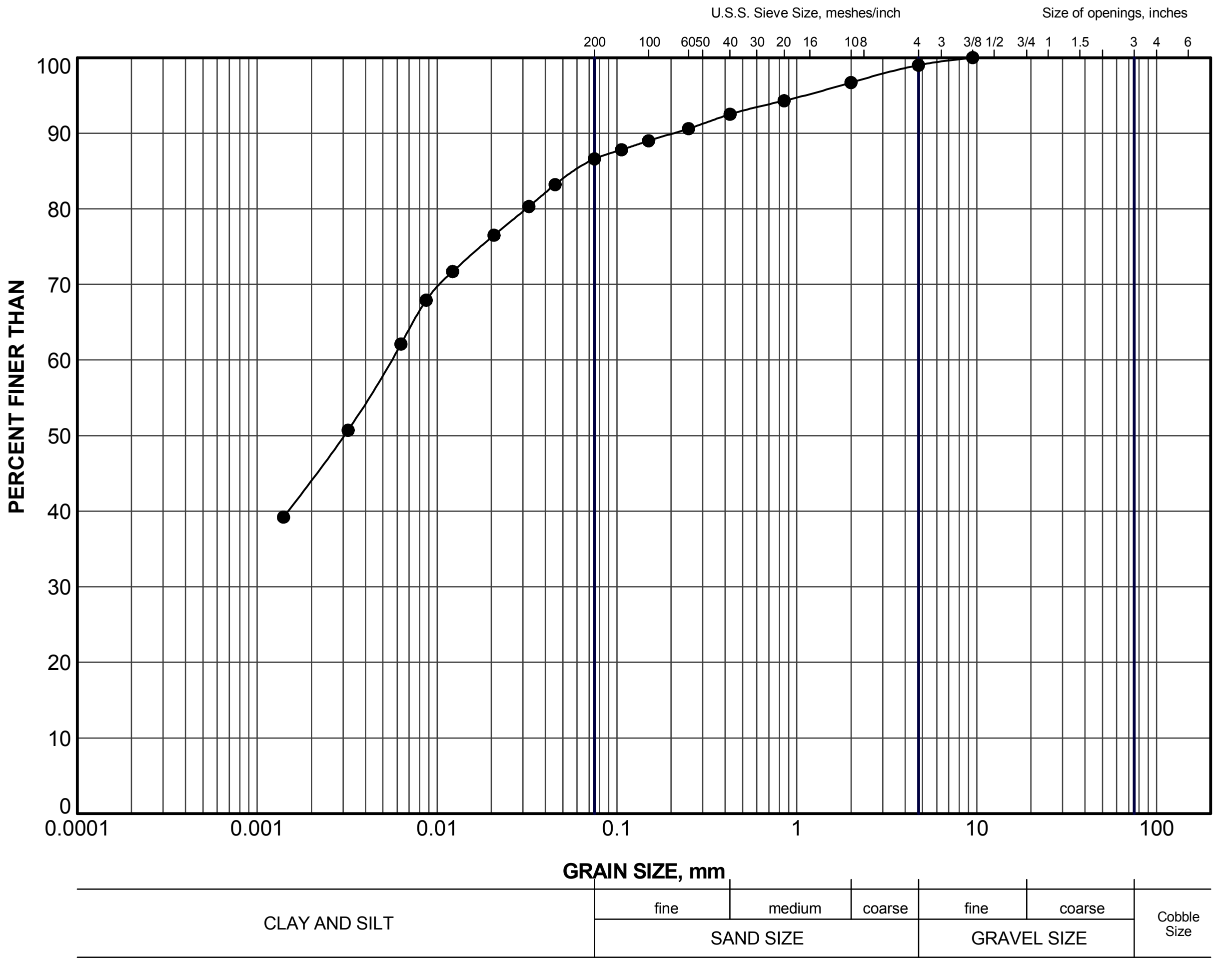
REGISTERED PROFESSIONAL ENGINEER F.J. HEFFERNAN Dec 16/15 PROVINCE OF ONTARIO

NO.	DATE	BY	REVISION

Geocres No. 40J8-66			
HWY. 401	PROJECT NO. 13-1132-0111		DIST.
SUBM'D. BT	CHKD. DUP	DATE: Dec. 9/15	SITE: 13-403/C
DRAWN: WDF	CHKD. WMK	APPD. FJH	DWG. 1

APPENDIX A

Laboratory Test Data



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	701	2	184.8

PROJECT
CULVERT No. 11, TEDFORD DRAIN, SITE 13-403/C
HIGHWAY 401/40 INTERCHANGE RECONFIGURATION
GWP 3093-09-00

TITLE

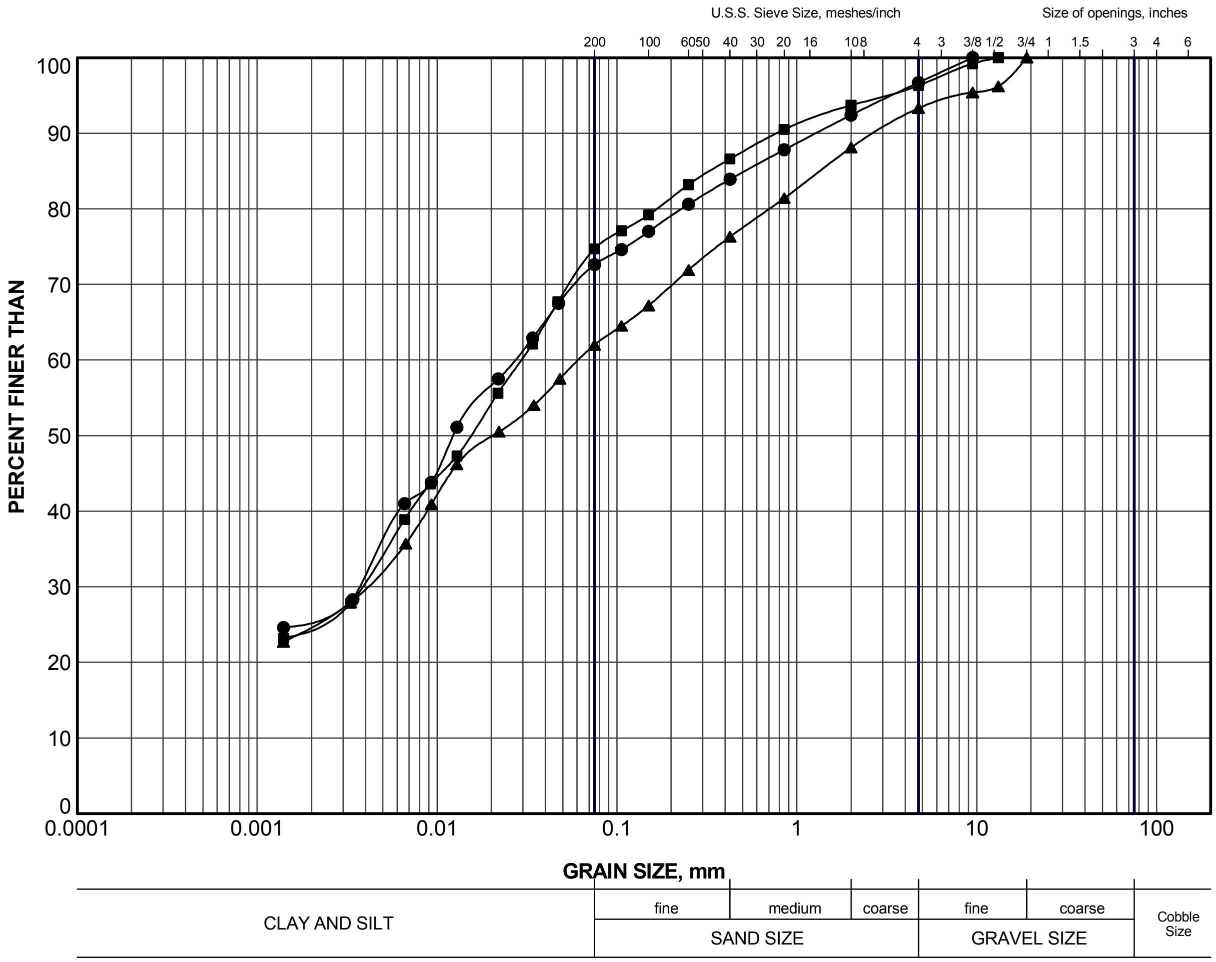
GRAIN SIZE DISTRIBUTION

SILTY CLAY TILL

Golder Associates

PROJECT No.		13-1132-0111	FILE No. 1311320111-1000-F110A1	
			SCALE N/A	REV.
DRAWN	WDF	Sep 25/15	FIGURE A-1	
CHECK				

LDN_MTO_GSD GLDR_LDN.GDT 25/09/15



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	701	6	181.7
■	702	4	183.6
▲	702	7	180.5

PROJECT
CULVERT No. 11, TEDFORD DRAIN, SITE 13-403/C
HIGHWAY 401/40 INTERCHANGE RECONFIGURATION
GWP 3093-09-00

TITLE

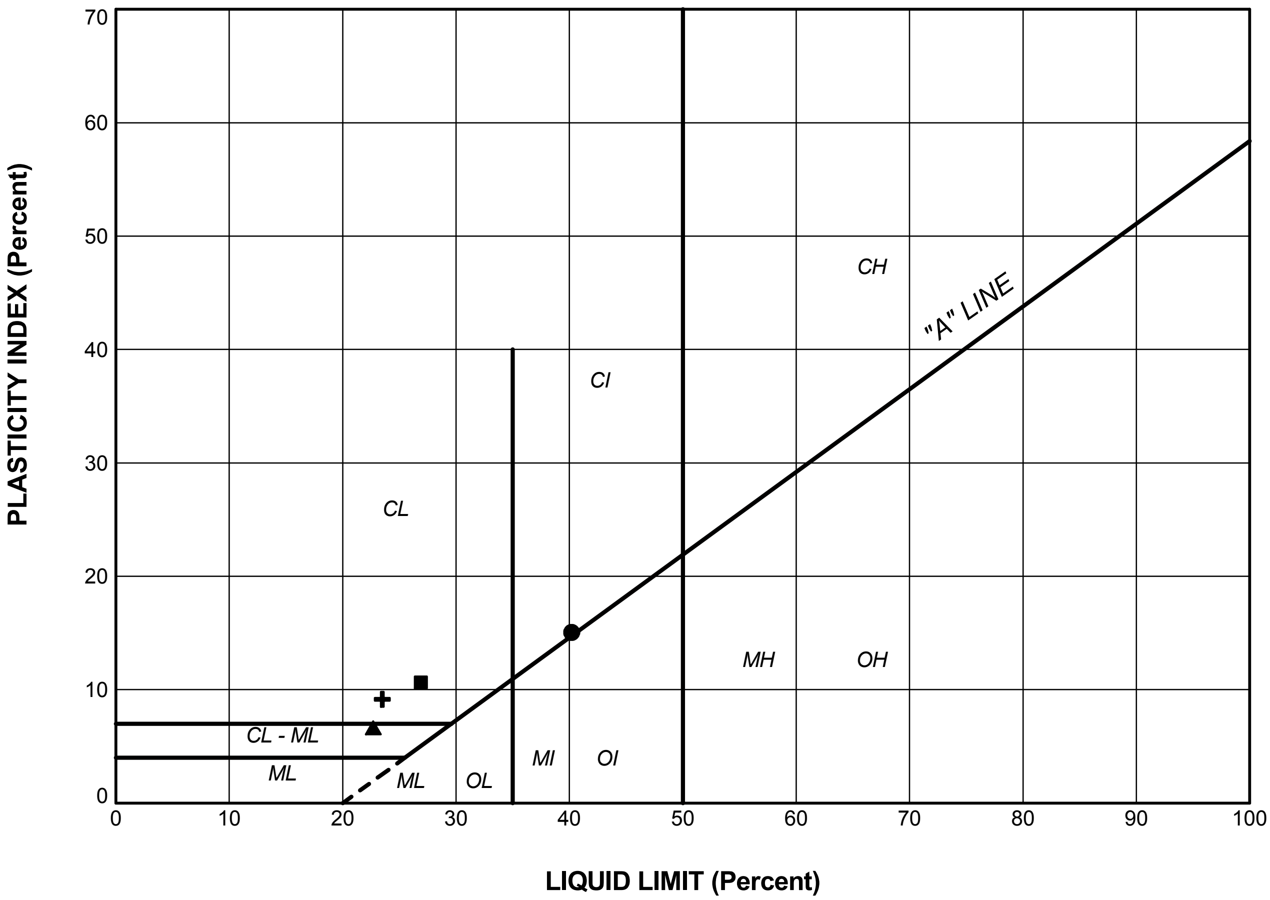
GRAIN SIZE DISTRIBUTION

CLAYEY SILT TILL

Golder Associates

PROJECT No.	13-1132-0111	FILE No. 1311320111-1000-F110A2	
		SCALE N/A	REV.
DRAWN	WDF	Sep 25/15	**FIGURE A-2**
CHECK			

LDN_MTO_GSD GLDR_LDN.GDT 25/09/15



SOIL TYPE
C = Clay
M = Silt
O = Organic

PLASTICITY
L = Low
I = Intermediate
H = High

LEGEND

SYMBOL	BOREHOLE	SAMPLE	LL(%)	PL(%)	PI
●	701	2	40.2	25.2	15.1
■	701	6	26.9	16.3	10.7
▲	702	4	22.7	16.1	6.7
✚	702	7	23.5	14.4	9.2

PROJECT CULVERT No. 11, TEDFORD DRAIN, SITE 13-403/C
HIGHWAY 401/40 INTERCHANGE RECONFIGURATION
GWP 3093-09-00

TITLE

PLASTICITY CHART

Golder Associates

PROJECT No.	13-1132-0111		FILE No. 1311320111-1000-F110A3	
			SCALE N/A	REV.
DRAWN	WDF	Sep 25/15	**FIGURE A-3**	
CHECK				

LDN_MTO_PI GLDR_LDN.GDT 25/09/15

APPENDIX B

Site Photographs

Photograph 1: South elevation (inlet) of Culvert Site 13-403/C.

Photograph 2: Highway 401 looking west from Culvert Site 13-403/C.

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