September 2013

FOUNDATION INVESTIGATION AND DESIGN REPORT

Culvert Replacement, Judges Creek Under Highway 6
Site No. 2-5/C, Station 14+320
Contract 1 Structure Replacements and Rehabilitation
GWP 3101-10-00
Ministry of Transportation, West Region

Submitted to:

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REPORT

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PART A

FOUNDATION INVESTIGATION REPORT

CULVERT REPLACEMENT, JUDGES CREEK UNDER HIGHWAY 6

SITE NO. 2-5/C, STATION 14+320

CONTRACT 1 STRUCTURE REPLACEMENTS AND REHABILITATIONS

GWP 3101-10-00

MINISTRY OF TRANSPORTATION - WEST REGION

1.0 INTRODUCTION

Golder Associates Ltd. (Golder Associates) has been retained by Stantec Consulting Ltd. (Stantec) on behalf of the Ministry of Transportation, Ontario (MTO) to carry out foundation investigations as part of the preliminary design and detail design work for GWP 3101-10-00. The project involves the detail design of the replacement and rehabilitation of several structures along multiple highways in Southern Ontario.

This report addresses the proposed replacement of the culvert at Judges Creek (Site 2-5/C) at Station 14+320 on Highway 6 just north of Edenhurst, Ontario in Bruce County.

The purpose of the foundation investigation is to explore the subsurface conditions at the location of the proposed structure replacement 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 and in Golder's proposal P2-1132-0163 dated February 25, 2013. The work was carried out in accordance with our Quality Control Plan for Foundation Engineering dated March 26, 2013.

2.0 SITE DESCRIPTION

The subject culvert is situated at Station 14+320 on Highway 6, approximately 1.7 kilometres south of Barrow Bay Road/Little Pike Bay Road in Bruce County, Ontario. The villages of Edenhurst and Ferndale are 0.3 kilometres south and 5.8 kilometres north of the site, respectively. The location of the culvert is shown on the Key Plan, Figure 1.

This section of Highway 6 is currently a two lane undivided highway with gravel shoulders. It is generally oriented north-south in the vicinity of the subject site. The creek flow direction in the culvert is from west to east beneath Highway 6. The existing culvert is a corrugated steel pipe (CSP) arch structure constructed in 1970 with the following characteristics:

Dimensions (m)	Obvert Elevation (m)		Construction
	Lt[1]	Rt[1]	
6.8 x 4.0 x 33.7	191.73	191.63	CSP Arch

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 drainage channel upstream and downstream of the culvert are grass covered and the channel flows through fields adjacent to Highway 6. Site photographs are provided in Appendix B.

The culvert is situated in a rural agricultural area with low relief. Ground surface elevations in the vicinity of the culvert site range from about 188 to 193 metres.

2.1 Site Geology

The project area is located within the Bruce Peninsula physiographic region. This region is characterized by very shallow soils with bare rock exposed in areas. Surficial silt deposits are mapped as covering most of the area in the vicinity of the site.[1] The quaternary geological mapping indicates that surficial soils consist of silt and clay with minor sand deposits.[2] Geological mapping also indicates that the underlying bedrock consists of light grey-tan to brown, bedded dolostone of the Guelph Formation of Middle to Lower Silurian age.[3] The bedrock surface at the site is at about elevations 171.5 to 172.7 metres, with the overburden thickness being about 17 to 18 metres.

[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] Barnett, P.J., Cowan, W.R. and Henry, A.P. 1991: Quaternary Geology of Ontario, southern sheet; Ontario Geological Survey, Map 2556, scale 1:1,000,000.

[3] Armstrong, D.K and Dubord, M.P., 1992: Paleozoic geology. Northern Bruce Peninsula, Southern Ontario; Ontario Geological Survey Map 198, Scale 1:50,000.

3.0 INVESTIGATION PROCEDURES

The field work for the investigation was carried out on June 5, 2013, 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 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 samplers used in the investigations 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.

Groundwater conditions in the boreholes were observed throughout the drilling operations and a standpipe piezometer was installed in borehole 101 as indicated on the corresponding Record of Borehole sheet. 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 experienced members 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.

In addition, information from the original subsurface investigation for the design of the existing structure was incorporated into this report. Data from boreholes 1 and 3 from Geocres Report No. 41A00-053 entitled "Foundation Investigation Report for Proposed New Bridge – Hwy. #6 and Judge's Creek, County of Bruce, Twp. of Eastnor, Lot 11, Con. II & III, District #5 (Owen Sound) W.J. 66-5-51 – W.P. 137-63" was used to supplement the current data.

The Record of Borehole sheets for previous boreholes are presented in Appendix C in their original format and annotated with metric conversions.

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)
101	4 976 551	402 448	193.1	12.7
102	4 976 530	402 445	193.0	6.6
1 (41A00-053)	4 976 543	402 435	189.6	20.5
3 (41A00-053)	4 976 543	402 454	189.9	19.0

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 pavement structure overlying variable embankment fill materials, or clayey silt and organics in the previous boreholes, then in sequence, clayey silt, silty gravel, and limestone bedrock.

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 Pavement Structure

Asphaltic concrete pavement was encountered at ground surface in boreholes 101 and 102. The pavement was 60 millimetres thick in both boreholes.

Pavement granular base materials were encountered beneath the asphalt in boreholes 101 and 102. The granular base materials were about 120 millimetres thick in both boreholes. Pavement granular subbase material was encountered beneath the granular base in borehole 102. The granular subbase material was about 340 millimetres thick.

4.2.2 Clayey Silt and Organics

Layers of clayey silt and organics were encountered at the ground surface in boreholes 1 and 3 (41A00-053). The surficial clayey silt and organic layers were about 1.5 to 1.7 metres in total thickness. Undrained shear strengths of 6 to 17 kilopascals (kPa) were obtained from in situ vane shear strength testing carried out in the clayey silt and organics indicating a very soft to soft consistency.

4.2.3 Fill

Sand fill materials were encountered beneath the pavement structure in boreholes 101 and 102 from elevations 192.9 to 192.5 metres. The sand fill materials were 1.6 to 5.3 metres thick with standard penetration test N[4] values of 2 to 17 blows per 0.3 metres. Samples of the sand fill had water contents above the groundwater level of about 7 to 9 per cent and below the groundwater level of about 15 to 16 per cent. Grain size distribution curves for samples of the fill materials are provided on Figure A-1.

A layer of clayey silt fill material was encountered beneath the sand fill material in borehole 102 at elevation 190.9 metres. The clayey silt fill material was 0.8 metres thick with an N value of 9 blows per 0.3 metres.

4.2.4 Clayey Silt

A stratum of very soft to stiff clayey silt was encountered beneath the fill in boreholes 101 and 102 at elevations 187.7 and 190.1 metres, respectively, and beneath the clayey silt and organics in boreholes 1 and 3 (41A00-053) at elevations 187.9 and 188.4 metres, respectively. Boreholes 101 and 102 were terminated in the clayey silt after exploring the layer for 3.7 to 7.2 metres. Where fully penetrated in boreholes 1 and 3 (41A00-053), the clayey silt was 15.1 to 15.7 metres thick. Measured N values for the clayey silt layers were 1 to 6 blows per 0.3 metres. In-situ vane shear strength testing indicated undrained shear strengths of 11 to 71 kPa indicating a very soft to stiff consistency. Water contents of the samples ranged from 20 to 41 per cent with an average water content of about 28 per cent. Based on a comparison of the 1966 and 2013 borehole data, it is likely that this clayey silt has gained strength by long-term consolidation beneath the 2 to 5 metres of road embankment fill.

The clayey silt is of low plasticity based on the Atterberg limits determination carried out on samples obtained during standard penetration testing. The average plastic limit was 19 per cent, the average liquid limit was 29 per cent, and the average plasticity index was 10 per cent. The Atterberg limits data for the clayey silt are presented on Figure A-3 and in Appendix C for Geocres 41A00-053. Grain size distribution curves for samples of the clayey silt are provided on Figure A-2.

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[4] The SPT 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 having first penetrated 150 millimetres.

4.2.5 Silty Gravel

Layers of silty gravel were encountered between elevations 172.2 and 173.3 metres beneath the clayey silt in boreholes 1 and 3 (41A00-053). The silty gravel was 0.5 to 0.7 metres thick. The silty gravel had a measured N value of greater than 100 blows per 0.3 metres and a water content of about 10 per cent.

4.2.6 Bedrock

Limestone bedrock was encountered beneath the silty gravel in boreholes 1 and 3 (41A00-053) at elevation 171.5 to 172.7 metres. Both boreholes were terminated in the limestone bedrock after exploring for 1.8 to 2.5 metres.

4.3 Groundwater Conditions

Groundwater conditions were observed during and on completion of drilling and sampling and a groundwater observation standpipe was installed in borehole 101. Installation details are provided on the corresponding Record of Borehole sheet following the text of this report. Groundwater was encountered in boreholes 101, 1, and 3 (41A00-053) at depths of 0.5 to 4.6 metres or between elevation 189.1 and 188.5 metres. A summary of the encountered and measured groundwater levels is provided in the table below.

Borehole	Ground Surface Elevation (m)	Encountered Groundwater Elevation (m)	Measured Groundwater Level Elevation (m)	
			June 5, 2013	September 12, 2013
101	193.09	188.5	188.47	188.64
102	193.03	dry	-	-
1 (41A00-053)	189.59	189.1	-	-
3 (41A00-053)	189.89	189.1	-	-

The above-noted encountered water levels are not considered to be representative of the long-term, stabilized groundwater conditions. The corresponding water level in the watercourse was measured at elevation 188.6 metres on June 5, 2013. On June 5, 2013 the water level in the groundwater observation standpipe installed in borehole 101 was about 4.6 metres below roadway surface or at about elevations 188.5 metres, respectively. A subsequent reading taken in the standpipe on September 12, 2013 was at about 4.5 metres below roadway surface at about elevation 188.6 metres.

Based on the observed groundwater levels, the surrounding topography, and water levels in the drain, the inferred groundwater level has been assumed to be elevation 188.5 metres for design purposes. 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 and will be influenced by flows in the watercourse.

5.0 MISCELLANEOUS

The investigation was carried out using equipment supplied and operated by Aardvark Drilling Inc., an Ontario Ministry of Environment licensed well contractor. The field operations were supervised by Mr. Michael Arthur under the direction of 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 and Ms. Nicole A. Gould, P.Eng. under direction of Mr. Azmi M. Hammoud, P.Eng. and reviewed by the Team Leader, Dr. Storer J. Boone, P.Eng. 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 REPLACEMENT, JUDGES CREEK UNDER HIGHWAY 6

SITE NO. 2-5/C, STATION 14+320

CONTRACT 1 STRUCTURE REPLACEMENTS AND REHABILITATIONS

GWP 3101-10-00

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 the replacement culvert at Judges Creek (Site 2-5/C), located at Station 14+320 on Highway 6 on the boundary of the Townships of Eastnor and Albemarle in Bruce County, Ontario.

The recommendations are based on interpretation of the factual data obtained from the current and previous boreholes advanced at this site. The interpretation and recommendations are intended to provide the designers with sufficient information to design the proposed culvert 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 26.0 metre long corrugated steel pipe (CSP) arch structure with a 6.8 metre span, a 4.0 metre high opening, and invert elevations of 187.7 and 187.6 metres at the inlet and outlet, respectively. The existing culvert has approximately 1.0 to 2.0 metres of fill cover.

6.2 Replacement Culvert

Based on information provided by Stantec Consulting Ltd. (Stantec), it is understood that consideration has been given to replace the existing CSP culvert with a new 6.8 by 4.0 metre CSP arch or a 6.0 by 3.5 metre pre-cast concrete box culvert. It has been indicated by Stantec that the pre-cast concrete box culvert is the preferred structural alternative as shown in the current preliminary general arrangement drawing provided by Stantec. Wingwall replacement will be in form of gabion basket walls or armor stone gravity walls. No grade raise is proposed at this location.

6.2.1 Foundations

The subsurface conditions encountered during the investigation generally consisted of the existing pavement structure overlying variable embankment fill materials and/or remnant clayey silt and organics, to elevations of between 187.7 and 190.1 metres, overlying the primary stratum of soft to stiff clayey silt to elevations of 172.2 and 173.2 metres. A thin layer of very dense silty gravel overlies the limestone bedrock which was encountered at elevations of 171.5 to 172.7 metres. The inferred groundwater level is at elevation 188.5 metres. The water level in the watercourse was about elevation 188.6 metres at the time of the investigation.

The culvert replacement should be designed to withstand the appropriate vertical weight of fill and traffic loading. It is not necessary to found a box culvert or a CSP arch at the standard depth for frost protection purposes as these types of structures are tolerant of small magnitude movements related to freeze-thaw cycles, should these occur. A box culvert or CSP arch should, however, be founded below any existing fill and surficial organic materials.

Based on the soil conditions encountered at the borehole locations, and assuming that the design culvert invert elevations will be similar to those of the existing culvert, the replacement box culvert or CSP arch may be founded on the soft to firm clayey silt at or below elevation 187.6 metres. Any observed fill or organic materials should be removed to the native soils. Any low areas should be brought to design grade using lean concrete fill or well graded granular materials.

Geotechnical Resistances

The soft to firm clayey silt is suitable for support of the proposed culvert replacement. A factored geotechnical resistance at Ultimate Limit States (ULS) of 150 kPa and a geotechnical resistance at Serviceability Limit States (SLS) of 100 kPa may be used for design purposes provided that any foundations have a minimum width of 1.0 metres and that the subgrade has been properly prepared (see Section 6.6). The SLS value corresponds to a maximum of 25 millimetres of total settlement for new culvert construction.

Frost Treatment and Scour Protection

Frost treatment in the form of a frost taper symmetrical about the culvert centreline must be provided in accordance with Ontario Provincial Standard Drawing (OPSD) 803.010 for a box culvert, or OPSD 802.020 for a CSP arch. The design frost penetration depth for this area is 1.4 metres below ground surface. The culvert base should be adequately protected against scour as noted in Section 1.9.5.2 of the Canadian Highway Bridge Design Code (CHBDC). Scour protection for the culvert backfill, bedding, and stream bank should be provided to protect the roadway, approach embankments, and culvert approaches.

Resistance to Lateral Forces/Sliding Resistance

The resistance to lateral forces/sliding resistance between the culvert base and the bedding or native soils should be calculated in accordance with Section 6.7.5 of the CHBDC.

In accordance with the CHBDC Section 6.7.5, a factor of 0.8 is applied in the equation to calculate the factored horizontal geotechnical resistance, H_{ri}, as follows:

$$H_{ri} = 0.8A'c' + 0.8V\tan\delta > H_f$$

where:

A'	-	effective contact area, square metres
c'	=	Nil
tan δ	-	as given below

V - unfactored vertical force, kilonewtons

H_f - unfactored horizontal load, kilonewtons

The coefficient of friction, tan δ, between the culvert base and founding material are shown in the following table.

Structure	Interaction	Angle of Friction, δ (degrees)	Coefficient of Friction, tan δ
CIP Box Culvert	CIP concrete on native clayey silt	24	0.44
Pre-Cast Box Culvert	Pre-cast concrete on Granular A bedding	30	0.58
CSP Arch	Corrugated steel pipe on Granular A bedding	27	0.51

6.2.2 Bedding

For pre-cast box culverts and CSP arches, bedding should be placed above a properly prepared subgrade from which all frozen, soft, uncompacted fill, organic materials, or other deleterious materials have been removed. Subexcavated material below the design subgrade elevation should be replaced with compacted Ontario Provincial standards Specifications (OPSS) Granular B Type II. It is recommended that the box culvert or CSP arch units be placed on a minimum thickness of 300 millimetres of Granular A bedding material and a minimum 75 millimetre thick levelling course consisting of uncompacted Granular A or fine aggregates as specified in MTO Special Provision (SP) 422S01.

6.2.3 Backfill and Cover

Backfill, cover, and construction of the frost taper (backfill transition) should be completed in accordance with OPSD 803.010 for a concrete box culvert, or OPSD 802.020 for a CSP arch. The excavation for the culvert replacement should exceed the culvert dimensions by at least one metre on each side to promote good workmanship and effective compaction of the fill.

The backfill should consist of free-draining, non-frost susceptible granular materials such as OPSS Granular A or Granular B Type III placed and compacted in accordance with OPSS 501 but with less than 5 per cent passing the No. 200 sieve. All bedding, backfill, and cover materials should be placed in accordance with OPSS 501 and 902 and SP 422S01.

Heavy compaction equipment should not be used immediately adjacent to the walls and roof of the culvert. The height of backfill adjacent to the culvert walls should be maintained equal on both sides of the structure during all stages of backfill placement with one side not exceeding the other by more than 500 millimetres.

Special care should be taken during backfilling and compaction of a CSP arch to ensure backfill is adequately compacted under the haunches.

6.3 Retaining Walls/Wingwalls

The existing structure has wingwalls that are about 2.5 metres long and 3.0 metres in height. It is understood that consideration has been given to constructing the wingwalls of the replacement structure as gabion or armour stone walls; however, the current preliminary general arrangement drawing provided by Stantec indicated that the replacement structure is to be designed without wingwalls. If wingwalls are to be constructed consideration may be given to utilizing gabion, armour stone, concrete gravity or cantilever, or reinforced soil system (RSS) walls. The various wingwall options are discussed below.

6.3.1 Wingwall Options

Armour Stone Walls

Construction of armour preparation and stone block walls at the site is geotechnically feasible. Armour Stone walls do not require an embedment depth equivalent to the frost depth provided they are founded on a granular pad of 300 millimetres thickness, and the founding row of stones has adequate embedment to provide a stable structure. Armour stone walls can be constructed relatively quickly as no foundation construction is required. Refer to Section 6.6 regarding subgrade placement of the granular pad.

Gabion Walls

Like armour stone 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 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, dissipation of the energy of flowing water, and they are free-draining provided an adequate filter is placed behind the wall. Gabion walls can be constructed relatively quickly with minimal equipment and materials. Refer to Section 6.6 regarding subgrade placement of the granular pad.

Reinforced Concrete Gravity and Cantilever Walls

Construction of reinforced concrete gravity or cantilever walls is geotechnically feasible. Compared to a concrete toe wall or RSS walls, footings for gravity and cantilever walls must be constructed with a frost cover of 1.4 metres. This may result in a longer foundation construction time compared to a gabion or armour stone wall, particularly if CIP walls are constructed. The concrete gravity wall could consist of pre-cast elements or CIP. Pre-cast wingwalls are preferred for compatibility with pre-cast culverts.

RSS Walls

The height of the wingwalls 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 system wall is selected, the geotechnical aspects of the global stability of the retaining wall design should be reviewed 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.

6.3.2 Foundations – Wingwalls

Armour stone walls may be founded on a 300 millimetre thick Granular A pad. A non-woven geotextile should be placed between the stone blocks and the backfill. Gabion walls may also be founded directly on a 300 millimetre thick Granular A pad. Non-woven geotextile is to be placed behind the gabions. Both the granular pad and the backfill should be placed in accordance with OPSS 512, OPSS 1860, and the manufacturer's specifications.

Cast-in-place reinforced concrete gravity and cantilever walls founded on concrete strip footings must be provided with a frost cover of 1.4 metres below the adjacent ground or thermal equivalent. Cast-in-place wingwalls should be kept structurally separate from a box culvert to accommodate some differential settlement.

Retained Soil System walls may be designed such that the facing blocks are constructed 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 1.4 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 to the top of the adjoining finished grade, is a minimum of 500 millimetres.

All wingwall foundations must be protected against scour as noted in the CHBDC Section 1.9.5.

Assuming the adjacent ground is at the existing culvert invert elevation of 187.6 metres, foundations for the various wall types may be founded on the native clayey silt at or below the elevations noted in the following table.

Wall Type	Armour Stone Block Walls	Gabion Walls	Concrete Gravity and Cantilever Walls	RSS Walls
Maximum Founding Elevation (m)	187.3	187.3	186.2	187.3

Wingwalls founded at or below the elevations indicated above may be designed using a factored geotechnical resistance at ULS of 150 kPa and a geotechnical reaction at SLS of 100 kPa. The SLS value corresponds to 25 millimetres of settlement.

If fill materials, organics, loose/softened soils, or otherwise deleterious materials are noted in the base of footing excavations, the excavations should be extended to the native soils. Any low areas of wingwall footings should be brought to design grade using lean concrete fill or compacted OPSS Granular A or Granular B Type II fill.

6.3.3 Resistance to Lateral Forces

The lateral pressures acting on the wingwalls will depend on the backfill soils, the type and method of placement of the backfill materials behind the walls, and the subsequent lateral movement of the structures. The resistance to lateral forces/sliding resistance between the wingwalls 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, angles of friction and corresponding unfactored coefficient of friction, tan δ, of 30 degrees and 0.56, respectively, may be used for the composite interaction between the base of the wall, levelling pad and the founding soil.

6.4 Liquefaction Potential and Seismic Analysis

6.4.1 Seismic Parameters

The site is located near the Town of Lion’s Head in southern Ontario. According to Table A.3.1.1 of the CHBDC, the zonal acceleration ratio, A, applicable to this site is 0.05. The corresponding acceleration related seismic zone, Z_a, is 1. Based on the site stratigraphy, the soil profile type is categorized as Type I with a seismic site response coefficient, S, of 1.0 based on the CHBDC criteria.

The importance category of the replacement culvert is “other” based on the current version of the CHBDC. The corresponding seismic performance zones (SPZ) to this importance category is 1. Structural culverts situated in SPZ 1 need not be analyzed for seismic loads. However, design forces for restraining elements and support lengths must meet the minimum requirements as outlined in CHDBC Clause 4.4.5.1. It should be noted that the MTO views culverts with spans greater than 3 metres as being similar to bridges. The designer should ensure that the selected culvert design meets the seismic requirements for buried structures as outlined in Clause 7.5.5 of the CHBDC.

6.4.2 Seismic Hazard Assessment

A preliminary screening of the soil stratigraphy was conducted using the procedure outlined in the Federal Highway Administration recommended procedures[5] and Canadian Foundation Engineering Manual (CFEM). The soils at this site are not considered to be susceptible to liquefaction. Therefore, a detailed evaluation of the liquefaction potential of the foundation soils is not considered warranted.

6.5 Lateral Earth Pressures for Design

The lateral pressures acting on the proposed culverts and associated wingwalls 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 II but with less than 5 per cent passing the No. 200 sieve should be used as backfill behind the walls. The fill should be compacted in loose lifts not greater than 200 millimetres in thickness. Longitudinal drains and weep holes should be installed to provide positive drainage of the granular backfill.
- A compaction surcharge equal to 12 kPa should be included in the lateral earth pressures for the structural design in accordance with CHBDC Figure 6.6.
- If the wall support does not allow lateral yielding (such is typically the case for a rigid concrete box culvert), 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 1.4 metres behind the culvert walls (case (a) from commentary on CHBDC Figure C6.6).
- For Case (a), the restrained case, which is typical for box culvert walls, the pressures are based on the existing embankment fill materials, assuming a Select Subgrade Material (SSM) is used, and the following parameters (unfactored) may be used:

Soil unit weight:	19 kN/m³
Coefficients of lateral earth pressure:	
'At rest' or restrained, K_o	0.53

[5] Federal Highway Administration (FHWA). (1997). "Design Guidance: Geotechnical Earthquake Engineering For Highways. Volume I – Design Principles." *Geotechnical Engineering Circular No. 3:FHWA-SA-97-076*, Washington, D.C.

- If the wall support allows lateral yielding (unrestrained structure, such as typically the case for wingwalls), 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 1.4 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 following parameters (unfactored) may be assumed:

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

6.6 Construction Considerations

6.6.1 General

Care should be taken during construction to avoid disturbance of the subgrades prior to constructing foundations for the replacement culvert and wingwalls. 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 and as modified by these recommendations.

It is recommended that the footing excavations be carried out such that the final 0.5 metres of excavation is completed with a geotechnical quality verification engineer (QVE) on site. Due to the relatively soft nature of the native cohesive soils, the last 0.5 metres of excavation should be carried out with a smooth-edge bucket to avoid creation of tooth gauges and soft areas. The prepared excavation bases should be inspected by the QVE and granular base materials or a working slab should be placed immediately after inspection to protect the founding materials. The appropriate non-standard special provisions (NSSPs) should be added to the contract documents to alert the contractor to the need to supply and place a working slab and use special procedures to carefully construct fills on the soft subgrade. Compacting granular materials placed directly on the relatively soft cohesive native soils will cause disturbance to both the native and granular fill materials. It is recommended that granular fill within 0.5 metres of the native soils be spread and lightly tamped with one to two passes of a walk-behind plate tamper. Heavy compaction to typical densities should be avoided for the first 0.5 metres of granular fill in these areas.

6.6.2 End Treatments and Camber

The culvert invert will be on the native clayey silt. At a minimum, an inlet end treatment in the form of a cut-off wall should be provided in accordance with Clause 1.9.5.6 of the CHBDC. However, a cut-off wall at the culvert outlet is not considered necessary. An outlet filter is not recommended due to the generally cohesive soils that will be present at the outlet, particularly if the head difference between the culvert inlet and outlet is low. No grade raising is proposed as part of the culvert replacement and relatively low cover is proposed for the replacement culvert; however, due to the presence of soft soils at the founding elevation, consideration should be given to providing a camber for the replacement culvert in order to accommodate differential settlement along the replacement culvert if a cast-in-place culvert is proposed.

6.6.3 Erosion and Scour Protection

Erosion and scour protection for the culvert inlet and outlet should be provided, as appropriate. Consideration could be given to using suitable non-woven geotextile and rip-rap, as required, to provide erosion protection based on hydraulic requirements. Rip-rap treatment at the culvert outlet should be provided in accordance with OPSD 810.010. In addition, sediment control such as silt fences and erosion control blankets may be required during construction.

6.7 Excavations and Groundwater Control

Excavations will extend through the existing pavement structure, fill, and organics and into the underlying clayey silt. It is anticipated that excavation for the culvert replacement will extend approximately 1.0 metres below the inferred groundwater level of elevation 188.5 metres. Some groundwater seepage from the native clayey silt should be anticipated. It is considered that groundwater can be controlled by pumping from properly constructed and filtered sumps located at the base of the excavations. Sumps should be maintained outside of the actual foundation 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 will need to be diverted/piped during construction. The appropriate NSSP should be included in the contract documents to alert the contractor about the need for adequate control of surface and groundwater flows.

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 fill materials and native very soft to firm clayey silt would be classified as Type 3 soils.

6.8 Staging and Temporary Roadway Protection

It is understood that a single lane is to remain open to traffic during construction; therefore, replacement of the existing culvert will need to be conducted in stages using a signalized single lane. Temporary support systems could consist of soldier piles and lagging or steel sheet piles. The temporary shoring may have a maximum height of 6 metres above the excavation base. Excavation support systems should be designed and constructed in accordance with OPSS 539 and the design should limit the lateral movement of the temporary shoring system to meet Performance Level 2. The contractor is responsible for the complete detailed design of the protection system.

Where the support to the wall is provided by anchors or rakers, the wall design should be based on a triangular earth pressure distribution using the design parameters given below. The raker/anchor support must be designed to accommodate the loads applied from pressures and surcharge pressures from area, line, or point loads as well as the impact of sloping ground behind the system. Passive toe restraint to the soldier piles may be determined using a triangular pressure distribution acting over an equivalent width equal to three times the pile socket diameter.

The unfactored triangular earth pressure distribution (p in kN/m^2; increasing with depth) can be calculated as follows:

$$p = K_a (\gamma H + q)$$

where H = the height of the excavation at any point in metres

K_a = active coefficient of earth pressure

γ = soil unit weight

q = surcharge for traffic and other loading

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		
Fill	0.36	0.53	2.8	28	19
Clayey Silt	0.33	0.50	3.0	30	20

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 Ms. Nicole A. Gould, P.Eng. under the direction Mr. Azmi M. Hammoud, P.Eng. and reviewed by the Team Leader, Dr. Storer J. Boone, P.Eng. 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.

GOLDER ASSOCIATES LTD.

ORIGINAL SIGNED

ORIGINAL SIGNED

Nicole A. Gould, P.Eng.

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Associate

ORIGINAL SIGNED

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MTO Designated Contact

NG/AMH/SJB/FJH/cr

n:\active\2012\1132 - geo\1132-0100\12-1132-0163 stantec-fdns-mega culverts-3011-e-0041\ph 1000-gwp 3101-10-00\rpts\r01\1211320163-1000-r01 sept 26 13 (final) part a&b fdns repl clvrt 2-5-c (judges creek).docx

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
SS	Split-spoon
DS	Denison type sample
FS	Foil sample
RC	Rock core
SC	Soil core
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.) split spoon 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) 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**	**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.

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
F	factor of safety
V	volume
W	weight

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
v	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

(a) Index Properties (continued)

w	water content
w_l	liquid limit
w_p	plastic limit
l_p	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_a	coefficient of secondary consolidation
m_v	coefficient of volume change
c_v	coefficient of consolidation
T_v	time factor (vertical direction)
U	degree of consolidation
σ'_p	pre-consolidation pressure
OCR	over-consolidation ratio $= \sigma'_p/\sigma'_{vo}$

(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 (ϕ = 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

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

Golder Associates
London, Ontario

RECORD OF BOREHOLE No 101

METRIC

PROJECT 12-1132-0163
W.P. 3101-10-00
LOCATION N 4976551.0 , E 402447.5
ORIGINATED BY MA
DIST
HWY 6
BOREHOLE TYPE POWER AUGER, HOLLOW STEM
COMPILED BY LMK
DATUM GEODETIC
DATE June 5, 2013
CHECKED BY

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa (+ FIELD VANE)	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
193.09	PAVEMENT SURFACE								
0.06	ASPHALT				Cold patch	193			
0.18	FILL, granular base Brown								
	FILL, sand, fine to coarse, trace to some silt, trace gravel Very loose to compact Brown	1	SS	5		192			
					Cuttings				
		2	SS	6		191			
		3	SS	11	Granular bentonite				
		4	SS	17				○	6 82 (12)
		5	SS	10	Filter sand	189			
		6	SS	2	Standpipe	188		○	
187.66 / 5.43	CLAYEY SILT, trace sand, with silt seams Firm to stiff Grey	7	SS	1		187		○ ○	
		8	SS	3				├┤ ○	0 3 73 24
						186	+2.1, +2.4		
		9	SS	6		185		○	
					Caved material	184	+2.3, +2.4		
		10	SS	3				├─┤ ○	0 2 80 18
						183	+2.0, +1.1		
		11	SS	4		182		○	
						181			
180.44		12	SS	2				○	
12.65	END OF BOREHOLE								

Groundwater encountered at about elev. 188.5m on June 5, 2013.

Water level measured in standpipe at elev. 188.47m after installation on June 5, 2013.

Water level measured in standpipe at elev. 188.64m on September 12, 2013.

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

Golder Associates
London, Ontario

RECORD OF BOREHOLE No 102

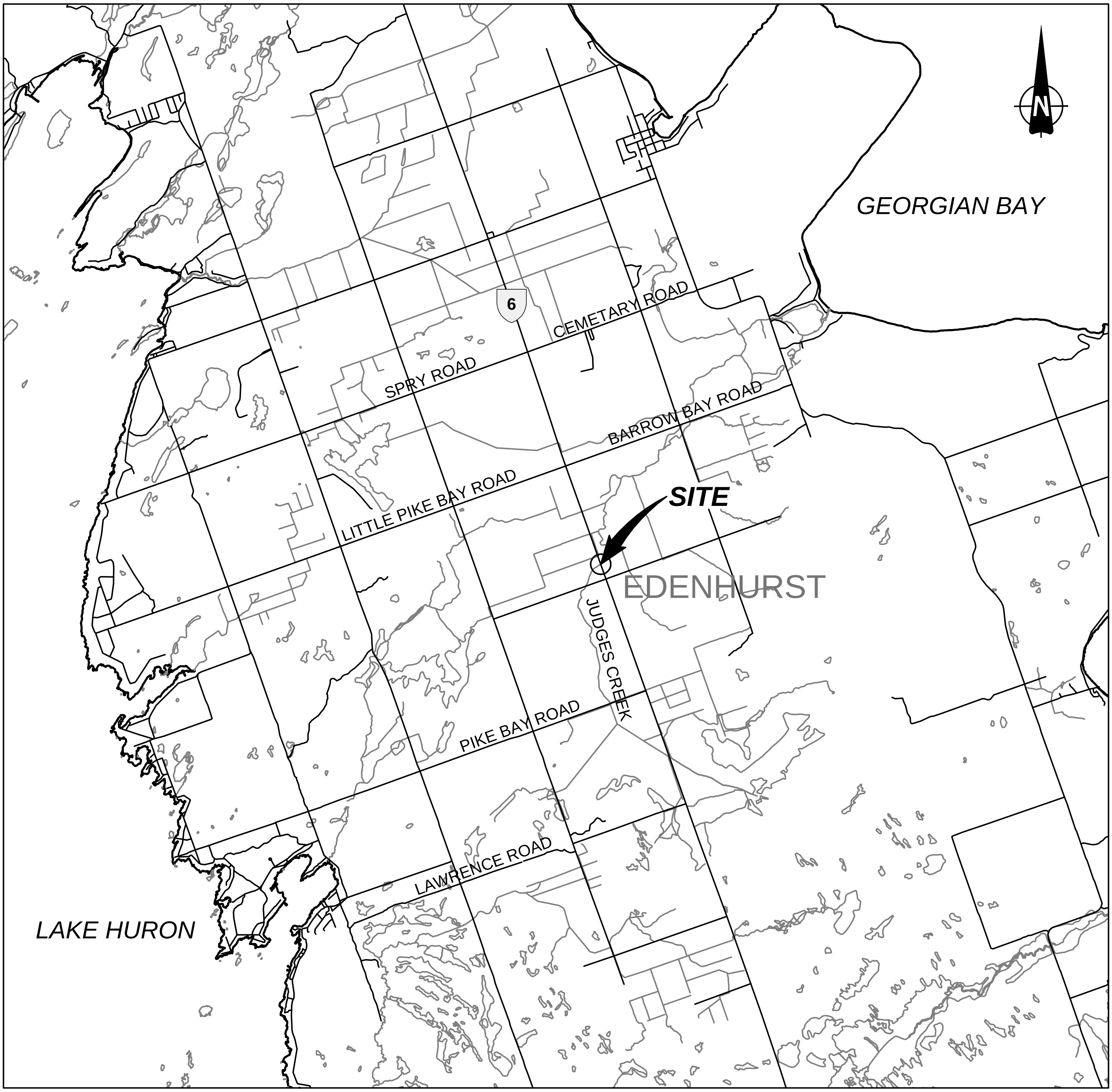
METRIC

PROJECT 12-1132-0163
W.P. 3101-10-00
LOCATION N 4976530.0 , E 402445.2
ORIGINATED BY MA
DIST
HWY 6
BOREHOLE TYPE POWER AUGER, HOLLOW STEM
COMPILED BY LMK
DATUM GEODETIC
DATE June 5, 2013
CHECKED BY

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	NATURAL MOISTURE CONTENT w (%)	GR	SA	SI	CL
193.03	GROUND SURFACE				193					
0.06	ASPHALT									
192.51 / 0.18	FILL, granular base, Brown									
0.52	FILL, granular subbase, Brown									
	FILL, sand, fine to medium, some silt, trace gravel, Compact, Brown	1	SS	17	192					
		2	SS	13		~10	2	86	(12)	
190.90 / 2.13	FILL, clayey silt, trace sand, trace gravel, trace topsoil, Stiff, Brown	3	SS	9	191					
190.13 / 2.90	CLAYEY SILT, trace sand, Soft to firm, Grey	4	SS	5	190					
		5	SS	6	189					
		6	SS	3	188	~27 (wP ~20, wL ~30)	0	1	74	25
		7	SS	3	187					
186.48 / 6.55	END OF BOREHOLE. Borehole dry during drilling on June 5, 2013.									

Column headings: SOIL PROFILE (ELEV DEPTH, DESCRIPTION, STRAT PLOT); SAMPLES (NUMBER, TYPE, "N" VALUES); 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

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



SCALE
0 2000 4000m
1:100,000

REFERENCE

PLAN BASED ON CANMAP STREETFILES V.2008.5.

NOTE

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

PROJECT

CULVERT REPLACEMENT, JUDGES CREEK
SITE No. 2-5/C, STATION 14+320
GWP 3101-10-00

TITLE

KEY PLAN

Golder Associates
LONDON, ONTARIO

PROJECT No.	12-1132-0163		FILE No.	1211320163-1000-F01001
			SCALE	AS SHOWN — REV. 0
CADD	LMK	July 22/13		
CHECK				FIGURE 1

Drawing file: 1211320163-1000-F01001.dwg Sep 24, 2013 - 4:56pm

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

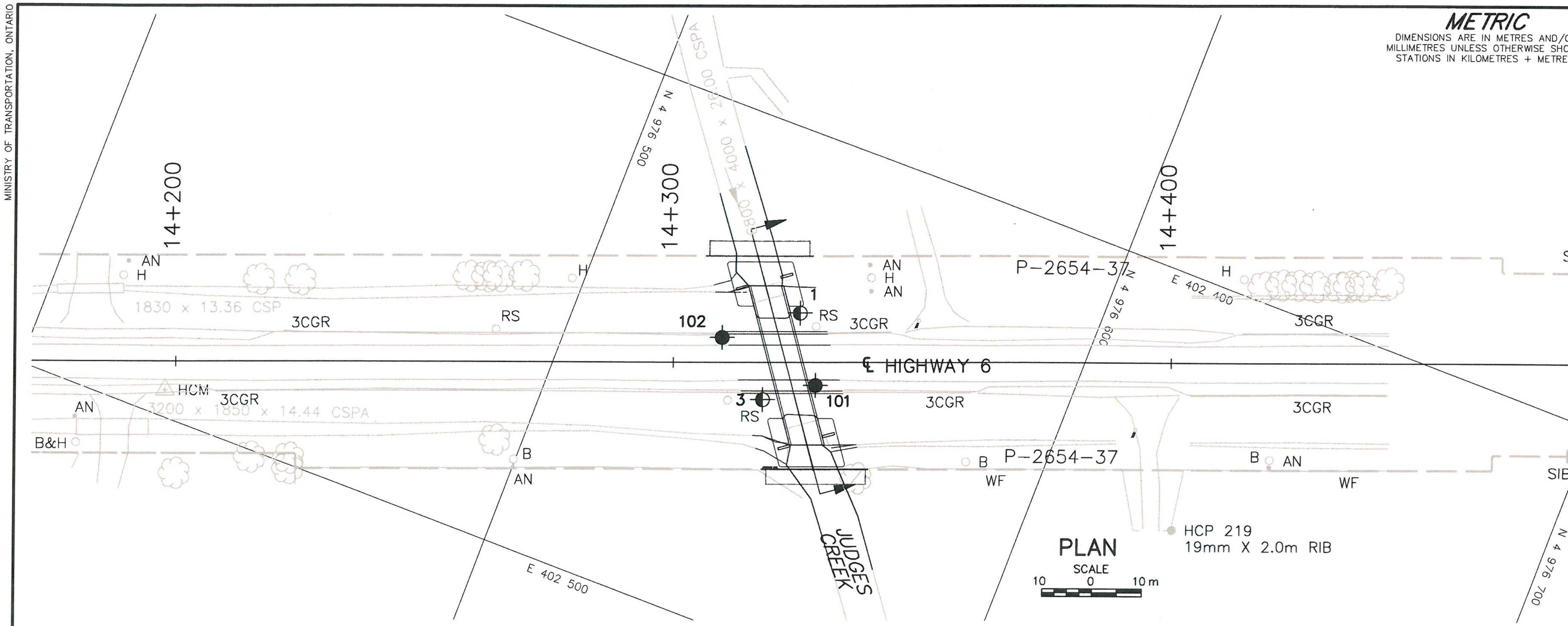
CONT No.
WP No. 3101-10-00

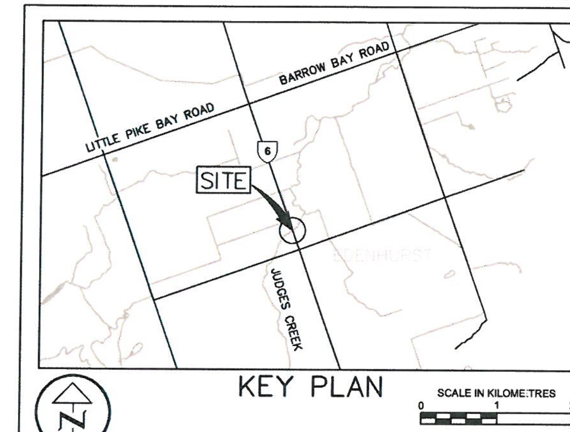
SHEET

CULVERT REPLACEMENT JUDGES CREEK

STRUCTURE REPLACEMENTS AND REHABILITATION
BORING LOCATIONS AND SOIL STRATA

Golder Associates Ltd.
LONDON, ONTARIO, CANADA

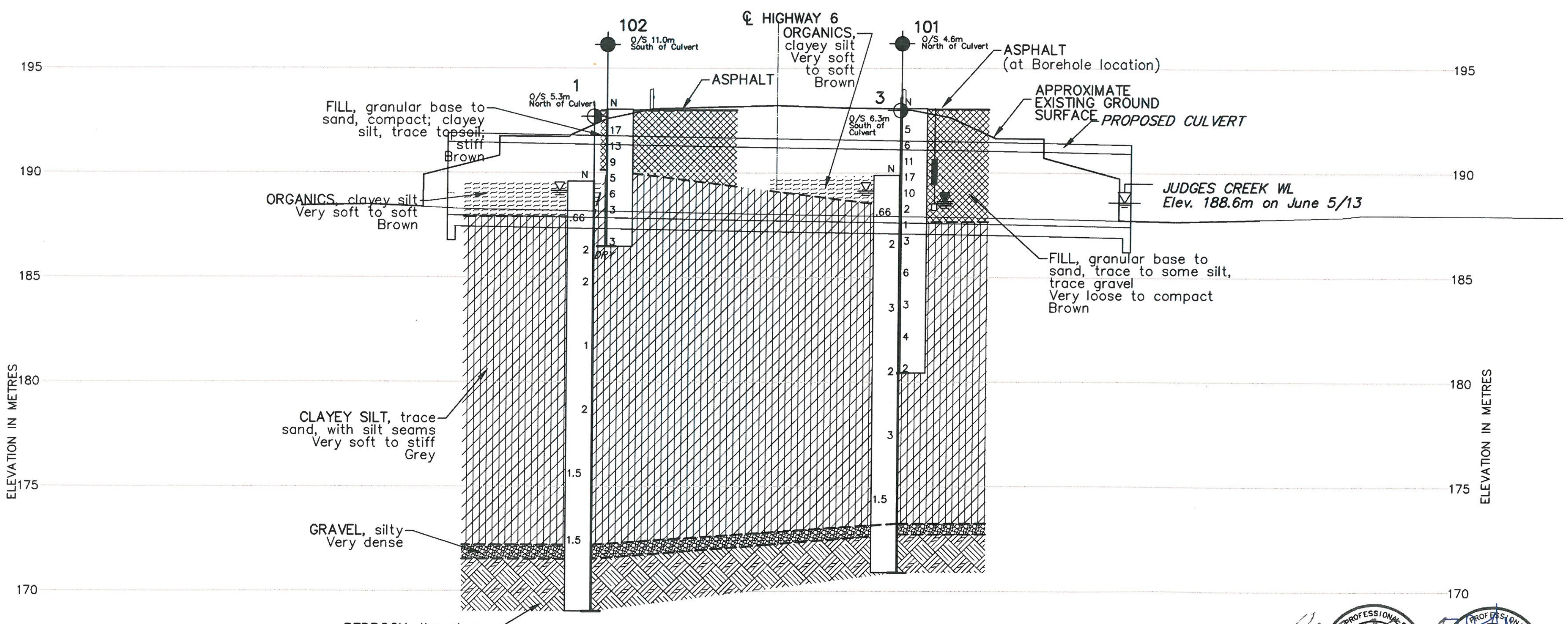




LEGEND

- Borehole – Current Investigation
- Borehole (Geocres 41A00-053)
- Seal
- Standpipe
- N — Standard Penetration Test Value
- 16 — Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- WL measured on June, 2013
- WL encountered during drilling
- DRY — Water level not established

No.	ELEVATION	CO-ORDINATES (MTM ZONE 10) NORTHING	EASTING
101	193.09	4 976 551.0	402 447.5
102	193.03	4 976 530.0	402 445.2
Geocres 41A00-053			
1	189.59	4 976 542.9	402 435.1
3	189.89	4 976 543.0	402 453.9



PROFILE ALONG ℄ CULVERT
HORIZONTAL SCALE 2.5 0 2.5m
VERTICAL SCALE 2.5 0 2.5m

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 by Stantec.

LICENSED PROFESSIONAL ENGINEER N. A. GOULD 100122043 PROVINCE OF ONTARIO

REGISTERED PROFESSIONAL ENGINEER F.J. HEFFERNAN PROVINCE OF ONTARIO

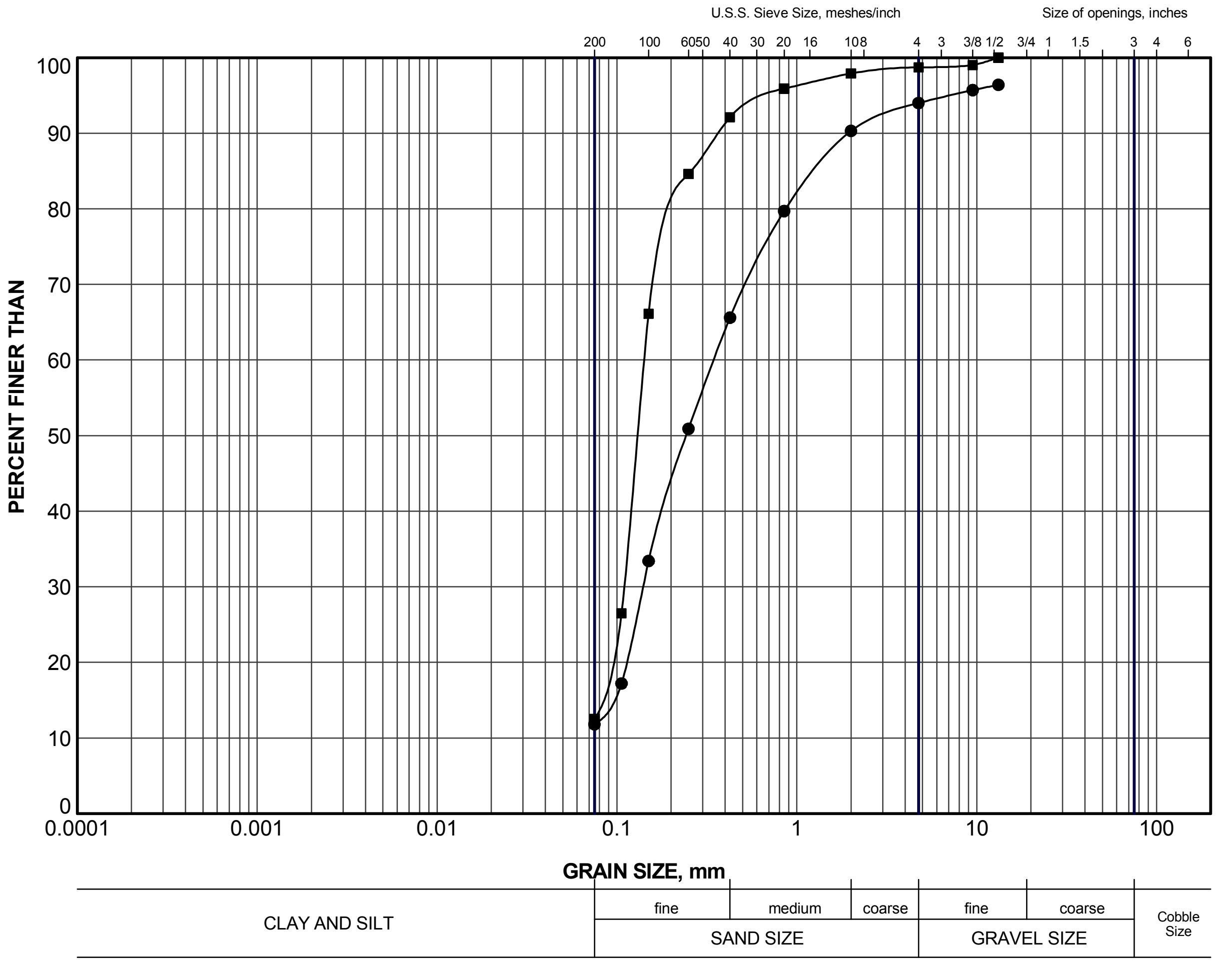
NO.	DATE	BY	REVISION

Geocres No. 41A-231			
HWY. 6		PROJECT NO. 12-1132-0163	DIST.
SUBM'D. BT	CHKD.	DATE: Sept. 20/13	SITE: 2-5/C
DRAWN: LMK	CHKD.	APPD.	DWG. 1

PLOT DATE: September 27, 2013
FILENAME: N:\Active\2012\1132 - Geo\1132-0100\12-1132-0163 STANTEC-FDNS-MEGA CULVERTS-3011-E-0041\Drafting\AutoCAD files\1211320163-1000-D01001.dwg

APPENDIX A

Laboratory Test Data



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	101	4	189.8
■	102	2	191.3

PROJECT

CULVERT REPLACEMENT, JUDGES CREEK
SITE No. 2-5/C, STATION 14+320
GWP 3101-10-00

TITLE

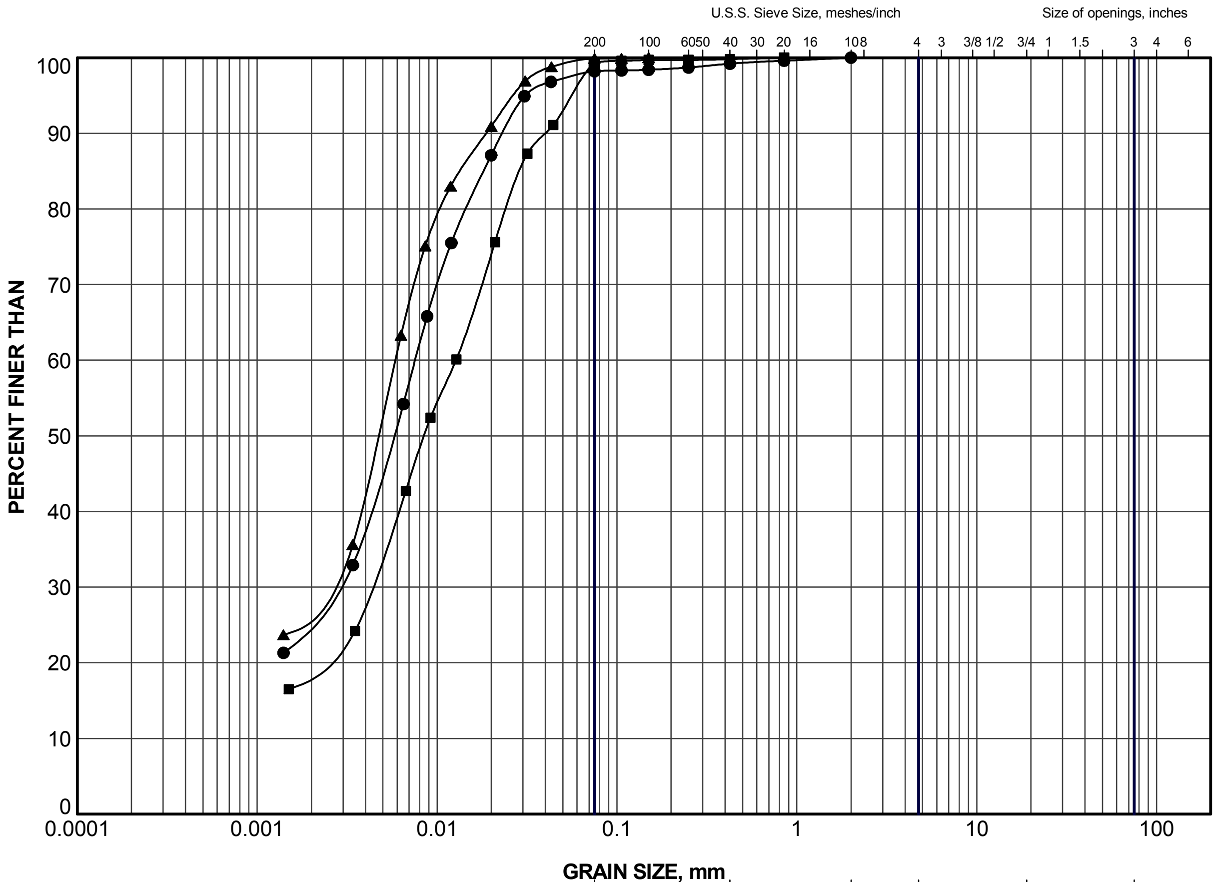
GRAIN SIZE DISTRIBUTION

SAND FILL

PROJECT No.		12-1132-0163	FILE No.	1211320163-1000-F010A1
			SCALE N/A	REV.
DRAWN	LMK	Jun 26/13	FIGURE A-1	
CHECK				

Golder Associates
LONDON, ONTARIO

LDN_MTO_GSD GLDR_LDN.GDT



CLAY AND SILT	SAND SIZE			GRAVEL SIZE		Cobble Size
	fine	medium	coarse	fine	coarse	

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	101	8	186.8
■	101	10	183.7
▲	102	6	188.2

PROJECT

CULVERT REPLACEMENT, JUDGES CREEK
SITE No. 2-5/C, STATION 14+320
GWP 3101-10-00

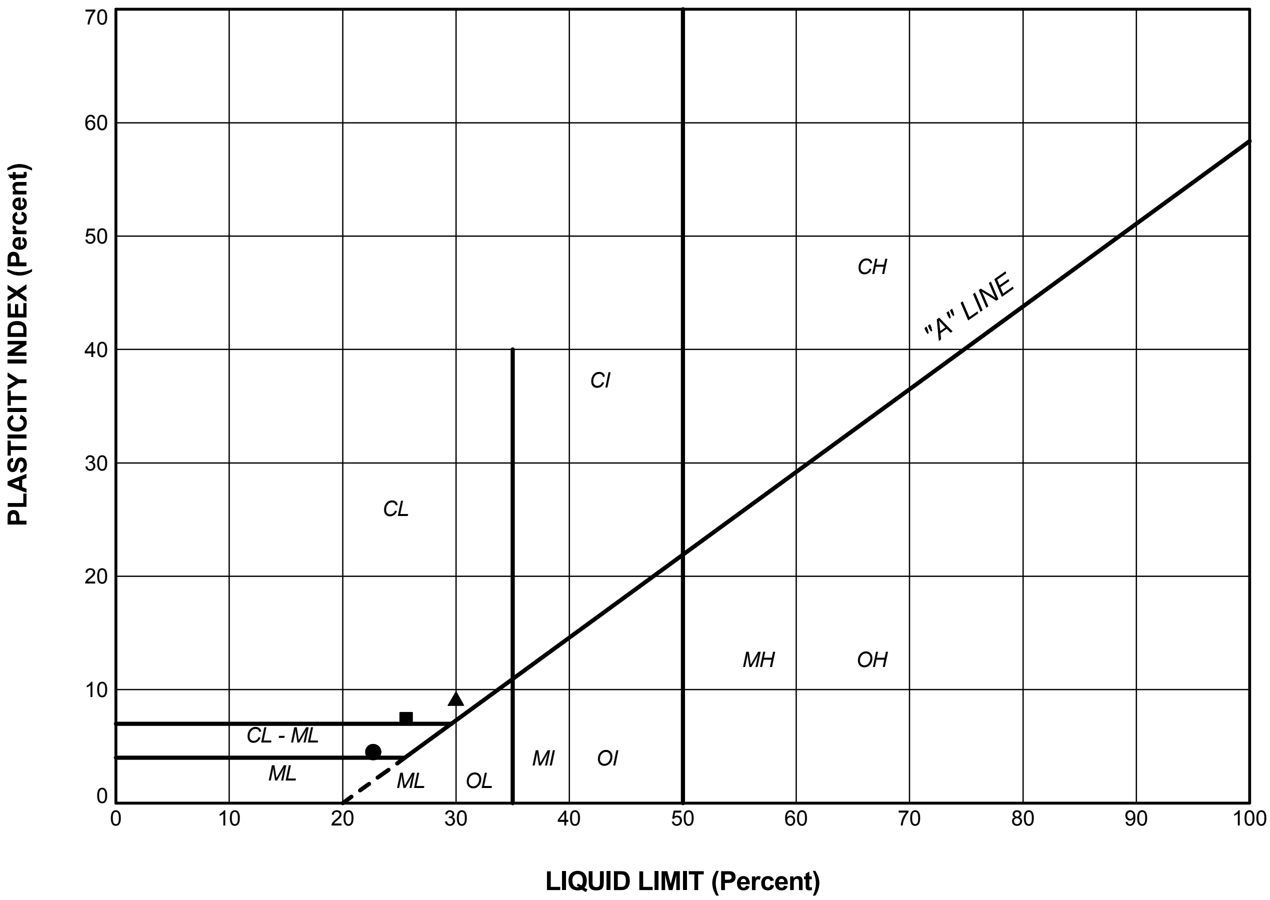
TITLE

GRAIN SIZE DISTRIBUTION

CLAYEY SILT

Golder Associates LONDON, ONTARIO	PROJECT No. 12-1132-0163		FILE No. 1211320163-1000-F010A2
			SCALE N/A REV.
	DRAWN LMK	Jun 26/13	FIGURE A-2
	CHECK		

LDN_MTO_GSD GLDR_LDN.GDT



SOIL TYPE
C = Clay
M = Silt
O = Organic

PLASTICITY
L = Low
I = Intermediate
H = High

LEGEND

SYMBOL	BOREHOLE	SAMPLE	LL(%)	PL(%)	PI
●	101	8	22.7	18.2	4.5
■	101	10	25.6	18.2	7.5
▲	102	6	30.0	20.8	9.2

PROJECT
CULVERT REPLACEMENT, JUDGES CREEK
SITE No. 2-5/C, STATION 14+320
GWP 3101-10-00

TITLE
PLASTICITY CHART

Golder Associates
LONDON, ONTARIO

PROJECT No.	12-1132-0163	FILE No.	1211320163-1000-F010A3	
		SCALE	N/A	REV.
DRAWN	LMK	Jun 26/13		
CHECK			FIGURE A-3	

LDN_MTO_PI GLDR_LDN.GDT

APPENDIX B

Site Photographs

Photograph 1: West elevation (inlet) of Culvert Site 2-5/C.

Photograph 2: East elevation (outlet).

Photograph 3: Highway 6 looking north from west shoulder at Culvert Site 2-5/C.

n:\active\2012\1132 - geo\1132-0100\12-1132-0163 stantec-fdns-mega culverts-3011-e-0041\ph 1000-gwp 3101-10-00\rpts\r01\1211320163-1000-r01 sept 26 13 (final) app b - photos.docx

APPENDIX C

Record of Borehole Sheets and Laboratory Test Results
Geocres Report Number 41A00-053

FORM DB-MT-126 (FORMERLY DB-ML-126)
68-1831

OFFICE REPORT ON SOIL EXPLORATION

(Geocres Report No. 4IA00-053)

"Note: This Drawing has been Reduced and is in Imperial Units - metric conversion also shown"

DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & TESTING DIVISION

RECORD OF BOREHOLE NO. 1

FOUNDATION SECTION

JOB 66-F-51
W.P. 137-63
DATUM 622.0

LOCATION Judges Creek & Hwy #6 Ch 141+85 - 25'-0" Lt.
BORING DATE ? May 25, 1966
BOREHOLE TYPE Washboring BX Casing

ORIGINATED BY W.W.K.
COMPILED BY W.W.K.
CHECKED BY

ELEV. DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	BLOWS / FOOT	REMARKS
622.0	189.59m (0.0m) Ground Level				
0.0	Dark Brown Clayey Silt & Organics Very Soft				Si70Cl30; W.L. 620.5 Observed In Casing
616.5 / 5.5	187.97m (1.68m) Clayey Silt Very Soft To Firm	1	SS	2/3	
		2	SS	2	
		3	SS	2	
		4	SS	1	Si.71Cl29
		5	SS	2	
		6	SS	1½	
		7	SS	1½	Sa.8 Si.64Cl.28
565.0	172.21m (17.38m) Silty Gravel V.Dense				
562.8 / 59.2	Limestone Bedrock 171.54m (18.04m)				
554.6 / 67.4	End Of Borehole 169.04m (20.54m)				

DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT: 10 20 30 40 50

SHEAR STRENGTH P.S.F.: 200 400 600 800 1000
+ - Field Vane (Borehole #1B)
● - Inconfined Compression (Borehole #1C)

LIQUID LIMIT w_L; PLASTIC LIMIT w_P; WATER CONTENT w

WATER CONTENT %: 15 30 45

BULK DENSITY γ P.C.F.

ELEV. SCALE: 620, 610, 600, 590, 580, 570, 560, 550

Field vane values (top to bottom): 00, 3.0, 2.0, 1.4, 1.7, 3.3, 4.5, 3.0, 3.5, 4.0, 4.0, 3.5, 3.0, 3.0, 2.4, 4.0, 3.5, 2.0, 1.4, 3.0, 2.0

100 per 8"

FORM OB-MT-126 (FORMERLY OB-ML-126)
65-1831

OFFICE REPORT ON SOIL EXPLORATION

(Geocres Report No. 4IA00-053)

"Note: This Drawing has been Reduced and is in Imperial Units - metric conversion also shown"

DEPARTMENT OF HIGHWAYS ONTARIO
MATERIALS & TESTING DIVISION

RECORD OF BOREHOLE NO. 3

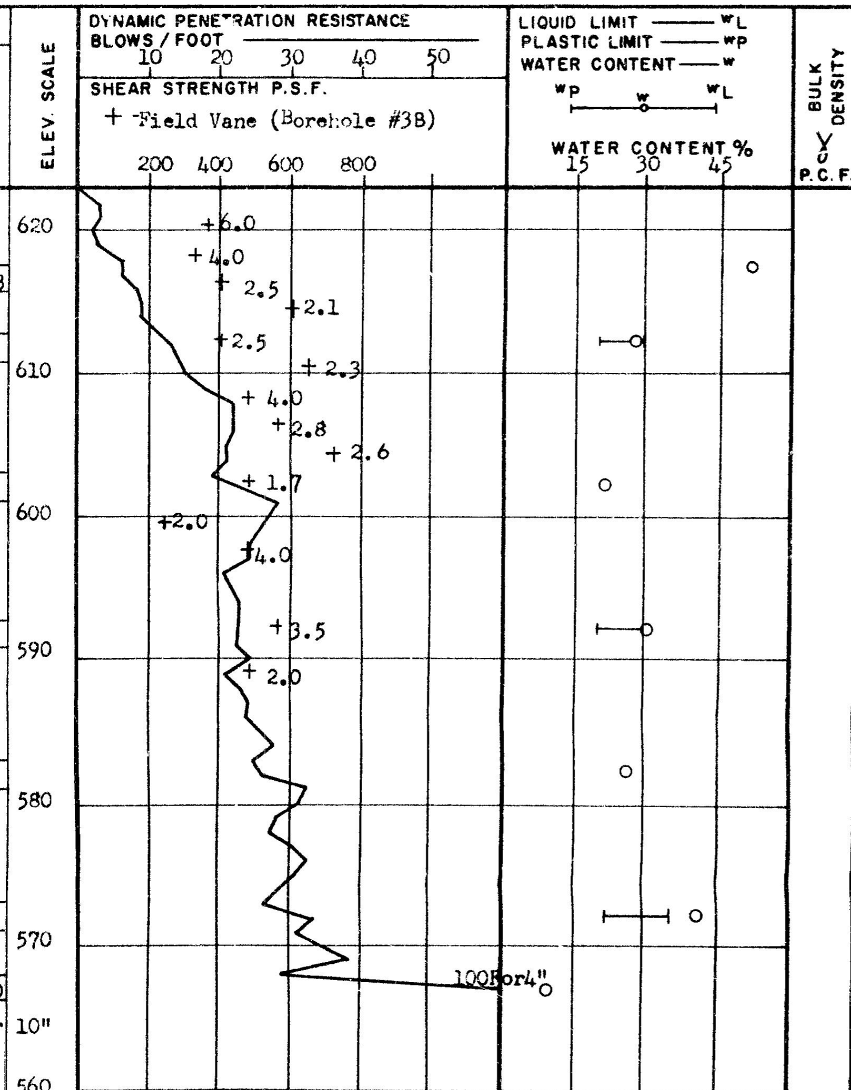
FOUNDATION SECTION

JOB 66-F-51
W.P. 137-63
DATUM 623.0

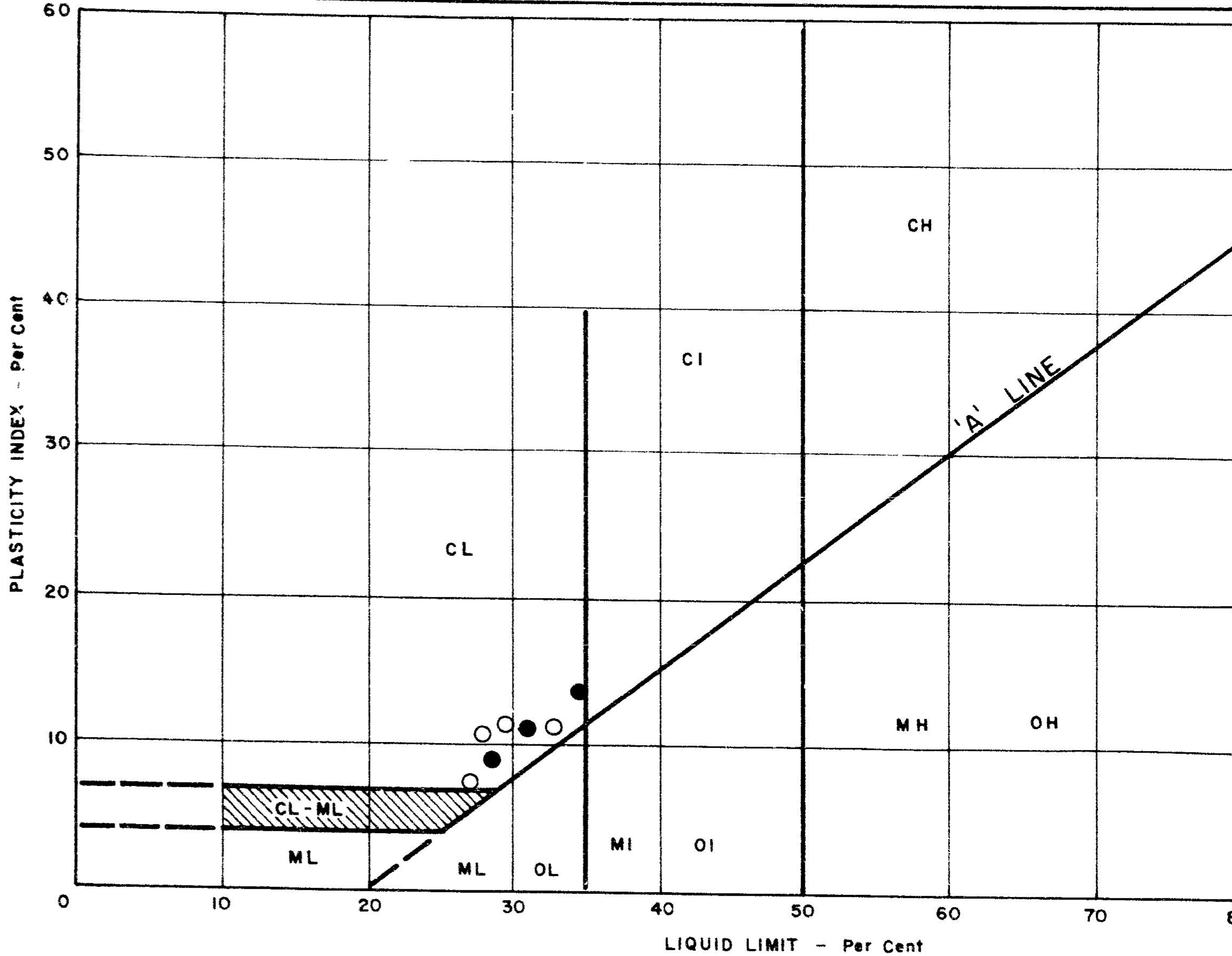
LOCATION Judge's Creek & Hwy #6 Ch 141+66 - 20'-0" Rt.
BORING DATE May 28, 1966
BOREHOLE TYPE Washboring BX Casing

ORIGINATED BY W.W.K.
COMPILED BY W.W.K.
CHECKED BY [illegible]

ELEV. / DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	BLOWS / FOOT	REMARKS
623.0	189.89m (0.0m) Ground Level				W.L. 620.5 Observed In Casing
0.0	Dark Brown Clayey Silt And Organics Soft				
618.0 / 5'-0"	188.37m (1.52m)	1	SS	2/3	
	CLAYEY SILT Very Soft To Firm	2	SS	2	Si.76Cl.24
		3	SS	3	
		4	SS	2	Si.65Cl.35
		5	SS	3	
		6	SS	1½	Si.60Cl.40
568.4	173.25m (16.64m)				
566.7	Silty Gravel - V. Dense	7	SS	130 For 10"	
56.3	172.73 (17.16m) Limestone Bedrock				
560.7 / 62.3	End of Borehole 170.90 (18.99m)				



FORM OB-MT-252
(REV. JUNE 1965)
65-1624



LEGEND		
BH No.	SAMPLE No.	SYMBOL
1		○
3		●

DEPARTMENT OF HIGHWAYS
MATERIALS and TESTING DIVISION
ONTARIO

PLASTICITY CHART

W.P. No. 137 - 63

JOB No. 66 - F - 51

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