October 2014

FOUNDATION INVESTIGATION AND DESIGN REPORT

Culvert Replacement, Washburn Drain
Highway 23, Site No. 25-331/C, Station 16+510
Contract 2 Structure Replacements and Rehabilitation
GWP 3040-11-00
Ministry of Transportation, West Region

Submitted to:

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REPORT

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Table of Contents

PART A - FOUNDATION INVESTIGATION REPORT

PART B - FOUNDATION DESIGN REPORT

APPENDICES

PART A

FOUNDATION INVESTIGATION REPORT

CULVERT REPLACEMENT, WASHBURN DRAIN
HIGHWAY 23, SITE NO. 25-331/C, STATION 16+510
CONTRACT 2 STRUCTURE REPLACEMENTS AND REHABILITATIONS
GWP 3040-11-00
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 detail design work for GWP 3040-11-00. The project involves the detail design of the replacement and rehabilitation of several culverts along multiple highways in Southern Ontario.

This report addresses the proposed replacement of the culvert at Washburn Drain (Site 25-331/C) at Station 16+510 on Highway 23 just south of Kirkton, Ontario in Perth 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 Associates' 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 16+510 on Highway 23, approximately 1.4 kilometres south of Kirkton Road in the Township of Blanshard, Perth County, Ontario. The villages of Kirkton and Woodham are 1.0 kilometres north and 0.6 kilometres south of the site, respectively. The location of the culvert is shown on the Key Plan, Figure 1.

This section of Highway 23 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 23. The existing culvert is a concrete non-rigid frame open footing (NRFO) structure constructed in 1950 with the following characteristics:

Dimensions (m)	Obvert Elevation (m)		Construction
	Lt[1]	Rt[1]	
3.66 x 1.83 x 21.03	299.62	299.59	NRFO

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 23. 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 300 to 304 metres.

2.1 Site Geology

The project area is located within the Stratford Till Plain physiographic region. This region is characterized by very shallow soils with bedrock exposed in areas of the region. 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 lacustrine silt and clay with some outwash silts.[2] Geological mapping also indicates that the underlying bedrock consists of medium brown microcrystalline limestone of the Dundee Formation of the Hamilton Group of Middle Devonian age.[3] The bedrock surface at the site is at about elevation 280.4 metres, with the overburden thickness being about 20 metres.[4]

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[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] Karrow, P.F., et al. 1974: Quaternary Geology of St. Mary's Area (Western Half), Southern Ontario; Ontario Division of Mines, Prelim. Map P.956, 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] Karrow, P.F. and Ferguson, A.J., 1975: Bedrock Topography of the St. Mary's Area. Southern Ontario; Ontario Division of Mines. Prelim. Map P.266, Scale 1:50,000.

3.0 INVESTIGATION PROCEDURES

The field work for the investigation was carried out on July 15 and 16, 2013, during which time 3 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). An automatic hammer was used.

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. In cases where it was not possible to achieve a full 450 millimetres of penetration, a penetration resistance representing the number of blows to drive the sampler a specific distance is recorded on the Record of Borehole sheets. The penetration resistance obtained in the first 150 millimetres is normally neglected unless the sampler could only be driven 150 millimetres or less, in which case SPT testing was terminated after 100 hammer blows. The results of the SPT testing as presented on the Record of Borehole sheets and Drawing 1 and discussed in Section 4 are unmodified (not standardized for hammer efficiency, borehole diameter, rod length, etc.). 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 226 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 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 (m)	Borehole Depth (m)
	Northing	Easting		
226	4 797 744	400 318	299.24	7.89
227	4 797 730	400 332	300.56	9.60
228	4 797 715	400 337	299.31	8.08

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 topsoil or fill at the ground surface overlying clayey silt, sandy silt till and clayey silt till.

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 layers 340 and 400 millimetres thick were encountered at the ground surface of boreholes 226 and 228, respectively. In addition, the cohesive fill in borehole 227 contained trace amounts of topsoil.

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 Fill

Sand and gravel material comprises the fill in borehole 227 at the ground surface to a depth of 0.6 metres. Cohesive fill was encountered at 0.6 metres depth in borehole 227 to a depth of 2.3 metres. The cohesive fill is firm to stiff with recorded N values of 6 and 11 blows per 0.3 metres. Samples of clayey silt fill had measured water contents of 20 to 21 per cent.

4.2.3 Clayey Silt

A stratum of firm to hard clayey silt was encountered beneath the topsoil in boreholes 226 and 228 at elevation 298.9 metres, and beneath the fill in borehole 227 at elevation 298.3 metres. The clayey silt layers were 1.2 to 2.7 metres thick. Measured N values for the clayey silt layers were 5 to 37 blows per 0.3 metres. Water contents of the samples ranged from 8 to 30 per cent.

The clayey silt is of low plasticity based on two Atterberg limits determinations carried out on samples obtained during standard penetration testing. The plastic limit was 15 per cent, the liquid limit varied from 24 to 26 per cent, and the plasticity index varied from 9 to 11 per cent. The Atterberg limits data for the clayey silt is presented on Figure A-4. Grain size distribution curves for samples of the clayey silt are provided on Figure A-1.

4.2.4 Sandy Silt Till

The clayey silt in all three boreholes was underlain by sandy silt glacial till from elevations 297.1 to 296.3 metres. Boreholes 226 and 228 were terminated in the sandy silt till after exploring the layer for about 5.0 to 5.6 metres. Where penetrated in borehole 227, the sandy silt till was 1.8 metres thick. The grain size distributions of four samples of sandy silt till are presented on Figure A-2. Cobbles were encountered within the sandy silt till in boreholes 226 and 227 and boulders, although not specifically encountered in the boreholes, should be anticipated in the sandy silt till due to its depositional history.

The sandy silt till is compact to very dense based on recorded N values of 14 to greater than 100 blows per 0.3 metres. The water contents of the sandy silt till samples varied between 9 and 11 per cent. The sandy silt till is of low plasticity based on a plastic limit of 12 per cent, liquid limit of 16 per cent and plasticity index of 5 per cent, based on one Atterberg limits determination. The results of the Atterberg limits testing are shown on Figure A-4.

4.2.5 Clayey Silt Till

Hard clayey silt glacial till was encountered beneath the sandy silt till in borehole 227 at elevation 295.2 metres. Borehole 227 was terminated in the clayey silt till after exploring the layer for about 4.3 metres. Cobbles were encountered within the clayey silt till in borehole 227 and boulders, although not specifically encountered in the boreholes, should be anticipated in the clayey silt till due to its depositional history.

The clayey silt till had recorded N values of 33 to greater than 100 blows per 0.3 metres. The water contents of the clayey silt till samples varied between 9 and 12 per cent. The clayey silt till is of low plasticity based on a plastic limit of 13 per cent, liquid limit of 26 per cent and plasticity index of 13 per cent, based on one Atterberg limits determination. The results of the Atterberg limits testing are shown on Figure A-4. The grain size distribution of a sample of clayey silt till is presented on Figure A-3.

4.3 Groundwater Conditions

Groundwater conditions were observed during and on completion of drilling and sampling and a groundwater observation piezometer was installed in borehole 226. Installation details are provided on the corresponding Record of Borehole sheet following the text of this report. Groundwater was encountered in boreholes 226 and 227 at depths of 2.1 to 3.7 metres or between elevation 297.1 and 296.9 metres during drilling. 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)			
			July 16, 2013	Sept. 4, 2013	Apr. 30, 2014	July 25, 2014
226	299.24	297.1	297.26	297.62	298.42	297.69
227	300.56	296.9	-	-	-	-
228	299.31	Dry to 291.2	-	-	-	-

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 Washburn Drain was measured at elevations 297.8 and 297.9 metres on July 16, 2013 and April 30, 2014, respectively. The reading on April 30, 2014 was taken after a heavy rainfall event.

Based on the observed groundwater levels, the surrounding topography, the soil colour change from brown to grey and water levels in the drain, the inferred groundwater level has been assumed to be elevation 298.0 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. Brett Thorner, E.I.T. under the direction of Mr. David J. Mitchell, the Site Investigation Field Manager. 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 direction of Ms. Dirka U. Prout, P.Eng. and reviewed by Mr. W. Michael Kellestine, P.Eng., a Principal 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.

GOLDER ASSOCIATES LTD.

ORIGINAL SIGNED

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PART B

FOUNDATION DESIGN REPORT

CULVERT REPLACEMENT, WASHBURN DRAIN
HIGHWAY 23, SITE NO. 25-331/C, STATION 16+510
CONTRACT 2 STRUCTURE REPLACEMENTS AND REHABILITATIONS
GWP 3040-11-00
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 Washburn Drain (Site 25-331/C), located at Station 16+510 on Highway 23 in of the Township of Blanshard in Perth County, Ontario.

The recommendations are based on interpretation of the factual data obtained from the 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 21.0 metre long concrete non-rigid frame open footing (NRFO) structure with a 3.7 metre span, a 1.8 metre high opening, and approximate invert elevations of about 297.8 metres. The existing culvert has approximately 1.5 to 2.0 metres of fill cover.

6.2 Replacement Culvert

Based on information provided by Stantec, it is understood that consideration is being given to replace the existing NRFO culvert with a new 4.8 by 2.4 metre concrete rigid frame box (RFB) culvert or a 4.8 by 2.4 metre concrete open footing culvert. The new culvert will be 21.35 metres long. It has been indicated by Stantec that a pre-cast concrete box culvert is the preferred structural alternative. No grade raise is proposed at this location. A precast box culvert is also the preferred alternative from a foundation engineering perspective. A comparison between open footing and box culvert options is provided in Table I.

It is proposed to replace the existing culvert using open cut installation methods. Traffic staging and temporary roadway protection will be required to facilitate the open trench installation of the replacement culvert to maintain a single lane of traffic during construction. Gabion walls with a typical length of 3.0 metres and height of 2.0 metres are proposed for each quadrant of the culvert.

6.2.1 Foundations

The subsurface conditions encountered during the investigation generally consisted of the topsoil or fill material to elevation 298 to 299 metres, underlain in sequence by firm to hard clayey silt to elevations between 296 and 297 metres then overlying compact to very dense sandy silt till. The sandy silt till generally extended to elevation 291 metres except in borehole 227 where hard clayey silt till was found beneath the sandy silt till from elevation 295 metres. The inferred groundwater level is at elevation 298.0 metres. The water level in the watercourse was about elevation 297.8 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 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 should, however, be founded below any existing fill and surficial organic materials.

The design culvert elevations at the inlet and outlet are 297.47 and 297.45 metres respectively. Based on the soil conditions encountered at the borehole locations, and the proposed design culvert invert elevations, the replacement box or open footing culvert may be founded on the very stiff clayey silt or compact sandy silt till. 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

Assuming a floor slab thickness of 300 millimetres for a box culvert and allowing for a minimum bedding thickness of 300 millimetres with a 75 millimetre thick levelling pad, the maximum founding elevation for a box culvert will be near elevation 296.8 metres. Allowing for a frost depth of 1.2 metres for open footing culvert foundations, the maximum founding elevation will be elevation 296.3 metres. A factored geotechnical resistance at Ultimate Limit States (ULS) of 350 kilopascals and a geotechnical reaction at Serviceability Limit States (SLS) of 230 kilopascals is recommended for foundations on the compact to very dense sandy silt till. These values assume that foundations have a minimum width of 0.5 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 or open footing culvert. The design frost penetration depth for this area is 1.2 metres below ground surface. Foundations for open footing culverts must be provided with a minimum frost cover equivalent to the frost depth or thermal alternative using insulating materials. 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
$\tan\delta$ - 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 or Open Footing Culvert	CIP concrete on native sandy silt till	32	0.62
	CIP concrete on native clayey silt	28	0.53
Precast Box Culvert	Pre-cast concrete on Granular A bedding/levelling pad	30	0.58

6.2.2 Bedding

For precast culverts, 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 Standard Specification (OPSS) Granular B Type II or Type III. It is recommended that the precast box 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 or open footing culvert. 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 0.075 millimetre 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.

6.2.4 End Treatments and Camber

The culvert will be founded on native compact sandy silt till or very stiff clayey silt. The use of outlet filters, clayey seals and the like are not considered necessary unless the hydraulic head difference between the culvert inlet and outlet is high. Box culverts are to be provided with cut-off walls at the inlet and outlet in accordance with Section 1.9.5 of the CHBDC. No grade raise is proposed as part of the culvert replacement and the cover is low. It is not necessary to provide a camber given that the foundation materials are such that significant differential settlement is not expected.

6.3 Gabion Retaining Walls

The proposed replacement culvert will feature gabion walls in each quadrant. Stantec has indicated that gabion walls are the preferred structural alternative given that the wall heights are less than 2 metres and the location is rural. From a geotechnical perspective, armour stone walls, concrete cantilever walls, concrete gravity walls, including precast toe walls, and reinforced soil system wall (RSS) are suitable alternatives. Considering wall heights of less than 2 metres, gabion walls, armour stone walls and precast concrete toe walls are more economical than RSS walls and concrete cantilever or other concrete gravity walls. Gabion walls, armour stone walls and RSS walls need not be founded at the frost depth and are the wall types most tolerant of movement. Although somewhat labour intensive to construct, installation of gabion walls will be more economical, rapid, and require less excavation and disruption to traffic than most other wall types. Gabion walls can be supported on the firm to very stiff clayey silt at or below elevation 298.0 metres. A factored geotechnical resistance of 275 kilopascals at ULS and a geotechnical reaction of 175 kilopascals at SLS may be used for design of the gabion wall footings. The SLS value corresponds to 25 millimetres of total settlement. If required, a granular levelling course approximately 75 millimetres in thickness can be placed on the founding strata for the gabion walls.

6.3.1 Frost Protection and Embedment

The frost depth applicable to this site is 1.2 metres. Gabion walls do not require an embedment depth equivalent to the frost depth provided they are founded on granular pads with a compacted thickness of 300 millimetres. In addition, the gabion walls should have sufficient embedment to provide stability and adequate protection against scour and erosion.

6.3.2 Lateral Resistance

The resistance to lateral forces/sliding resistance between the underside of the gabion baskets and levelling pads or subgrade soil, as applicable, should be calculated in accordance with Section 6.7.5 of the CHBDC. Also, the retaining walls shall be checked for overturning. The following angles of friction and corresponding unfactored coefficient of friction, tan ϕ', may be used for the interaction between the gabion baskets and the founding soil:

Subgrade Material	Angle of Friction, ϕ' (degrees)	tan (ϕ')	K_p
Clayey silt	28	0.53	2.8
Granular A levelling pad	30	0.58	3.0

In accordance with the CHBDC, a factor of 0.8 is to be applied in calculating the horizontal resistance. The factored horizontal geotechnical resistance, H_{rs}, should be based on CHBDC 6.7.5 as follows:

$$H_{rs} = 0.8A'c' + 0.8V\tan\phi' > H_f$$

Where:

A' - effective contact area, square metres
c' = 0
δ = angle of interface friction
V - unfactored vertical force, kilonewtons
H_f - factored horizontal load, kilonewtons

The unfactored coefficients of passive pressure, K_p, for the portion of the retaining walls below the ground surface are given in the above table using unfactored effective angles of internal friction, ϕ'.

6.3.3 Other Design Considerations

The gabion walls must incorporate surface drainage measures to minimize infiltration of surface water into the backfill behind the wall. It is recommended that a drainage swale be incorporated at the top of each wall with the flow directed to a positive outlet. Free draining backfill must be used behind the walls. An approved non-woven geotextile should be placed at the rear of these walls in order to minimize clogging and or loss of fines through the gabion stone. The gabion walls should be designed and constructed in accordance with SP 512S03.

6.4 Liquefaction Potential and Seismic Analysis

6.4.1 Seismic Parameters

The site is located near the Town of Exeter in southern Ontario. According to Table A.3.1.1 of the CHBDC, the zonal acceleration ratio, A, applicable to this site is 0.00. The corresponding acceleration related seismic zone, Z_a, is 0. 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 of 3 metres or greater 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 culvert and 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. 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.
- 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 1.2 metres behind the culvert walls (case (a) from commentary on CHBDC Figure C6.20).

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[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.

- 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

- If the wall support allows lateral yielding (unrestrained structure, such as typically the case for the 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 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

6.6 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. 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 Quality Verification Engineer (QVE) experienced in geotechnical engineering on site. The prepared excavation bases should be inspected by the QVE and granular base materials or a working slab, where appropriate, should be placed immediately after inspection to protect the founding materials. Based on the results of borehole 228, clayey silt may be exposed in the vicinity of the outlet if the replacement box culvert is founded near elevation 296.3 metres. The clayey silt is susceptible to disturbance from foot traffic and excessive uptake of moisture. If a clayey silt subgrade is exposed and the granular bedding or foundation cannot be placed within 24 hours of excavation then it must be protected with a working slab. The working slab is to consist of a minimum 100 millimetre thick mat consisting of concrete with a minimum 28 day compressive strength of 10 megapascals. A Non Standard Special Provision (NSSP) should be added to the Contract

Documents to provide the working slab requirements. The QVE should assess the foundation conditions to determine if sub-excavation of unsuitable material is required. Sub-excavation and placement and compaction of fill should be carried out under the direction of the QVE.

6.6.2 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 in accordance with OPSS 805.

6.7 Excavations and Groundwater Control

Excavations will extend through the existing fill materials and topsoil and into the underlying clayey silt and sandy silt till. It is anticipated that excavation for the culvert replacement will extend approximately 2.0 metres below the inferred groundwater level of elevation 298.0 metres. Some groundwater seepage from the fill materials and native clayey silt and sandy silt till 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.

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 would be classified as a Type 3 soils and the native clayey silt and sandy silt till would be classified as Type 2 soils.

6.8 Staging and Temporary Roadway Protection

It is understood that a single lane is to remain open to traffic during construction with one half of the culvert being replaced at a time. Temporary support systems, installed at or near the centre line of the highway could consist of soldier piles and lagging or steel sheet piles. 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 cantilevered sheet piles are used, or 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:

$$p' = K_a (H - h_w)\gamma + K_a(\gamma - \gamma_w)h_w + \gamma_w h_w + K_a q$$

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

K_a = active coefficient of earth pressure

γ = soil unit weight

γ_w = unit weight of water or 9.8 kN/m^3

q = surcharge for traffic and other loading

h_w = height of groundwater level above excavation base/water level to be taken as elevation 298.0 metres

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
Sandy Silt 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 reviewed by Mr. W. Michael Kellestine, P.Eng., a Principal 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.

GOLDER ASSOCIATES LTD.

ORIGINAL SIGNED ORIGINAL SIGNED

Dirka U. Prout, P.Eng. Project Engineer	Fintan J. Heffernan, P.Eng. MTO Designated Contact

BT/DUP/WMK/FJH/cr

n:\active\2012\1132 - geo\1132-0100\12-1132-0163 stantec-fdns-mega culverts-3011-e-0041\ph 2000-gwp 3040-11-00\rpts\r04 - site 25-331 (washburn)\1211320163-2000-r04 oct 9 14 (final) part a&b fdns repl clvrt 25-331-c (washburn).docx

TABLE I

COMPARISON OF STRUCTURE ALTERNATIVES FOR REPLACEMENT CULVERT

Washburn Drain, Site 25-331, Station 16+510, Highway 23
Structure Replacements and Rehabilitation
GWP 3040-11-00

FOUNDATION OPTION	FEASIBILITY	ADVANTAGES	DISADVANTAGES	ESTIMATED COSTS	RISKS/ CONSEQUENCES
Precast box culvert founded on very stiff clayey silt or compact sandy silt till	• Feasible • Preferred technical alternative	• Least expensive option due to shallower excavation compared to the open footing option and use of precast elements • Allows for most rapid construction compared to the two other alternatives since there is no wait for concrete to cure • Suitable for corrosive environments • Can be more readily installed during cold weather conditions	• If floor is thin and poorly reinforced, it may crack and heave. • During high flows, the concrete floor can be undermined • Susceptible to defects/leakage at joints	• Low	• Relatively low risk • Improper alignment or transition between stream and culvert may lead to problems with scour
CIP box culvert founded on very stiff clayey silt or compact sandy silt till	• Feasible	• Intermediate in cost between a pre-cast box and CIP open footing culvert • Less excavation required compared to an open footing culvert • Suitable for corrosive environments • Culvert design can be customized in the field for high stress or load conditions or other site-	• If floor is thin and poorly reinforced, it may crack and heave. • During high flows, the concrete floor can be undermined • More expensive compared to precast box option due to increased labour associated with concrete formwork • Special curing	• Low to Moderate	• Relatively low risk • Improper alignment or transition between stream and culvert may lead to problems with scour

COMPARISON OF STRUCTURE ALTERNATIVES FOR REPLACEMENT CULVERT

FOUNDATION OPTION	FEASIBILITY	ADVANTAGES	DISADVANTAGES	ESTIMATED COSTS	RISKS/ CONSEQUENCES
		specific requirements • Can be constructed with far fewer joints than a precast box culvert	requirements for cold weather work		
CIP open footing culvert with spread footings founded on very stiff clayey silt or compact to very dense sandy silt till	• Feasible	• Higher costs compared to box culvert options • Suitable for corrosive environments • Culvert design can be customized in the field for high stress or load conditions or other site-specific requirements • Can be constructed with far fewer joints than a precast box culvert • Most suitable where maintenance of fish and/or wildlife passage and preservation of the natural stream bed is a priority	• More expensive compared to both box culvert options due to increased labour associated with concrete formwork and deeper excavation required to provide frost protection for footings • Footings may be susceptible to scour and undermining especially at entrance	• Moderate	• Relatively low to moderate risk • Improper alignment or transition between stream and culvert may lead to problems with scour • Option with foundations most susceptible to damage by scour

NOTES:
1. Qualitative estimates are based on 2014 construction costs and are intended to provide a comparison between alternatives rather than actual construction costs.
2. Table to be read in conjunction with accompanying report.

Prepared By: BT
Checked By: DUP

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

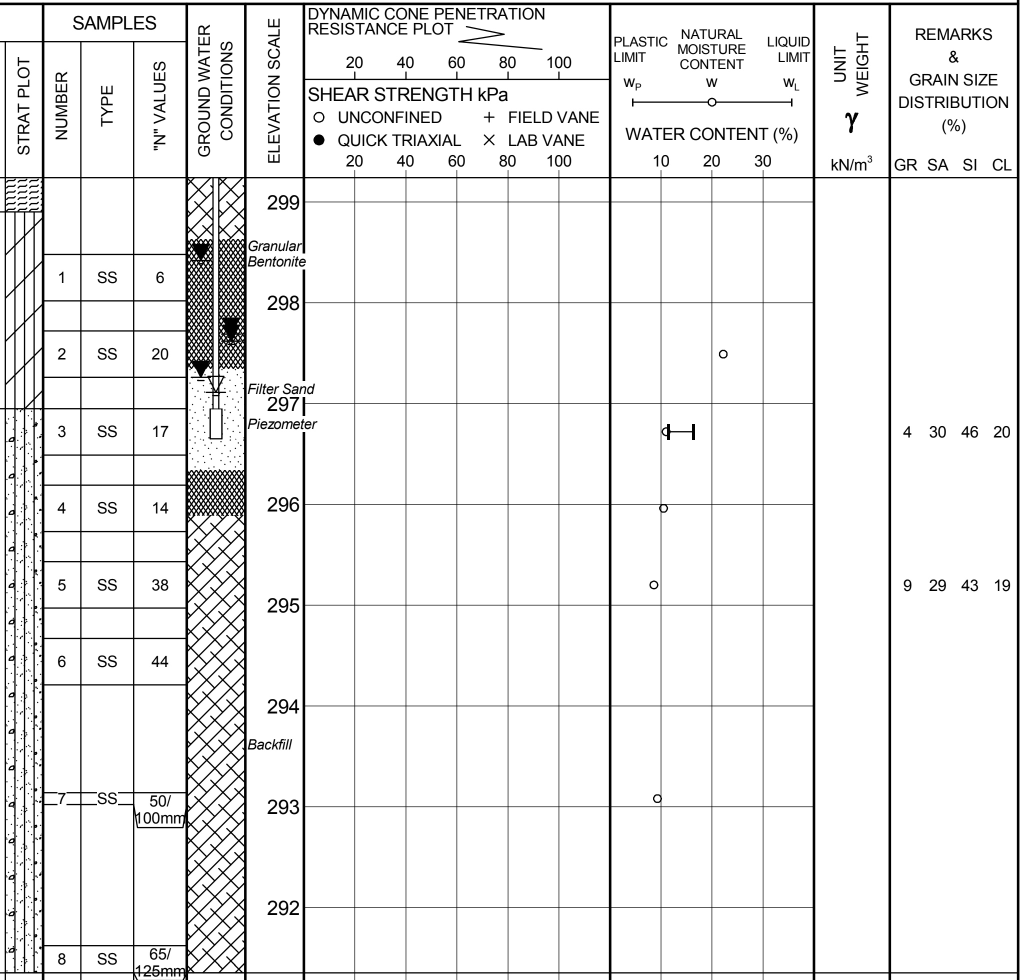
Golder Associates
London, Ontario

RECORD OF BOREHOLE No 226

1 OF 1 **METRIC**

PROJECT 12-1132-0163
W.P. 3040-11-00
LOCATION N 4797743.6 , E 400317.5
ORIGINATED BY BT
DIST ____ HWY 23
BOREHOLE TYPE POWER AUGER, HOLLOW STEM
COMPILED BY AMG/LMK
DATUM GEODETIC
DATE July 15, 2013 - July 16, 2013
CHECKED BY ____

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR	SA	SI	CL
299.24	GROUND SURFACE							
0.00	TOPSOIL, silty Brown							
298.90 0.34	CLAYEY SILT, trace to some sand Firm to very stiff Brown and grey becoming brown at about elev. 297.9m	1	SS	6				
		2	SS	20				
296.95 2.29	SANDY SILT TILL, some clay, trace to some gravel, with cobbles Compact to very dense Brown becoming grey at about elev. 296.3m	3	SS	17	4	30	46	20
		4	SS	14				
		5	SS	38	9	29	43	19
		6	SS	44				
		7	SS	50/ 100mm				
		8	SS	65/ 125mm				
291.35 7.89	END OF BOREHOLE							



Groundwater encountered at about elev. 297.1m during drilling on July 15, 2013.

Water level measured at 297.26m on July 16, 2013.

Water level measured at elev. 297.62m on September 4, 2013.

Water level measured at elev. 298.42m on April 30, 2014.

Water level measured at elev. 297.69m on July 25, 2014.

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

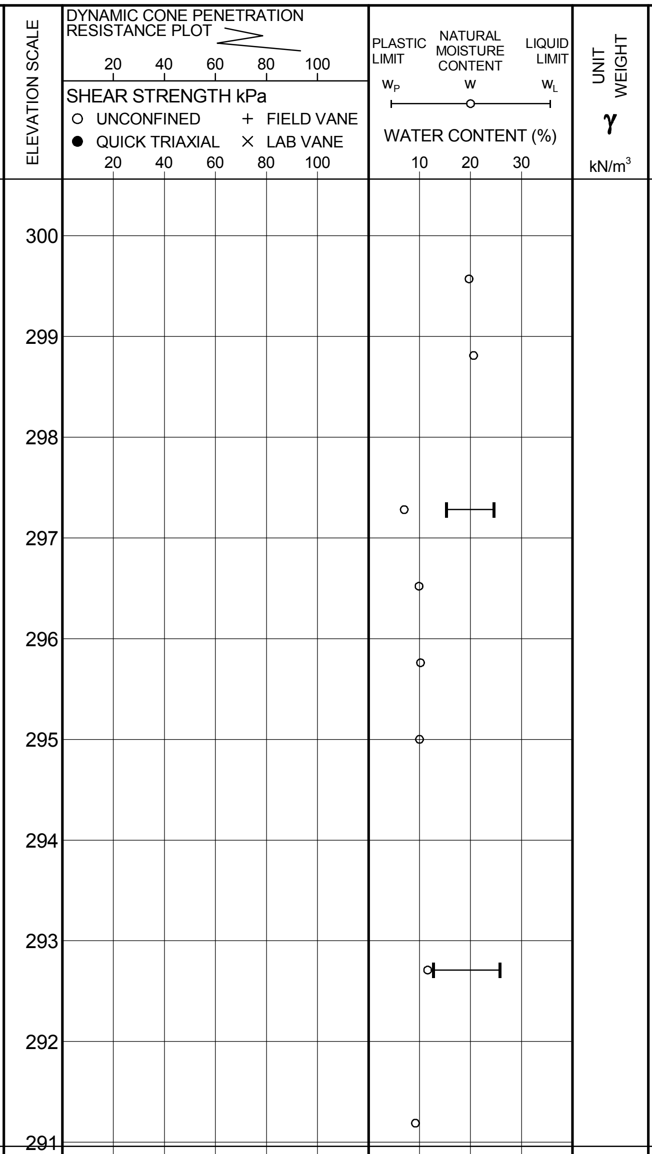
Golder Associates
London, Ontario

RECORD OF BOREHOLE No 227

1 OF 1 **METRIC**

PROJECT 12-1132-0163
W.P. 3040-11-00
LOCATION N 4797730.2 , E 400331.8
ORIGINATED BY BT
DIST
HWY 23
BOREHOLE TYPE POWER AUGER, HOLLOW STEM
COMPILED BY AMG/LMK
DATUM GEODETIC
DATE July 16, 2013
CHECKED BY

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLES NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR	SA	SI	CL
300.56	GROUND SURFACE								
0.00	FILL, sand and gravel, some silt, with cobbles Brown								
300.01 0.55	FILL, clayey silt, some sand, trace gravel, topsoil Firm to stiff Brown	1	SS	11					
		2	SS	6					
298.27 2.29	CLAYEY SILT, trace sand, gravel Stiff to hard Brown and grey	3	SS	8					
		4	SS	37		2	7	56	35
297.05 3.51	SANDY SILT TILL, some clay, trace gravel, with cobbles Compact Brown becoming grey at about elev. 296.1m				▽				
		5	SS	18					
		6	SS	20		4	31	45	20
295.23 5.33	CLAYEY SILT TILL, some sand, trace gravel, with cobbles Hard Grey	7	SS	33					
		8	SS	76/ 225mm					
		9	SS	53		2	26	44	28
		10	SS	103					
290.96 9.60	END OF BOREHOLE Groundwater encountered at about elev. 296.9m during drilling on July 16, 2013.								



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

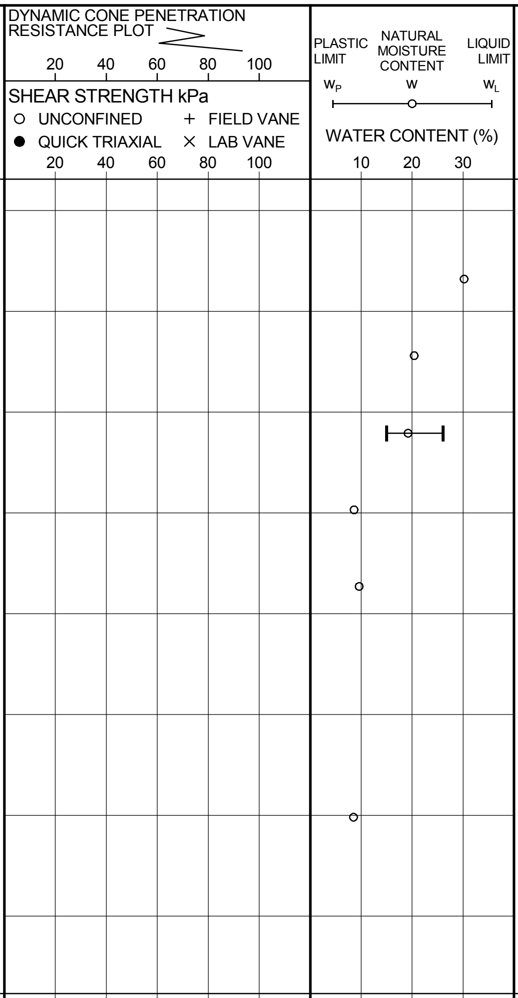
Golder Associates
London, Ontario

RECORD OF BOREHOLE No 228

1 OF 1 **METRIC**

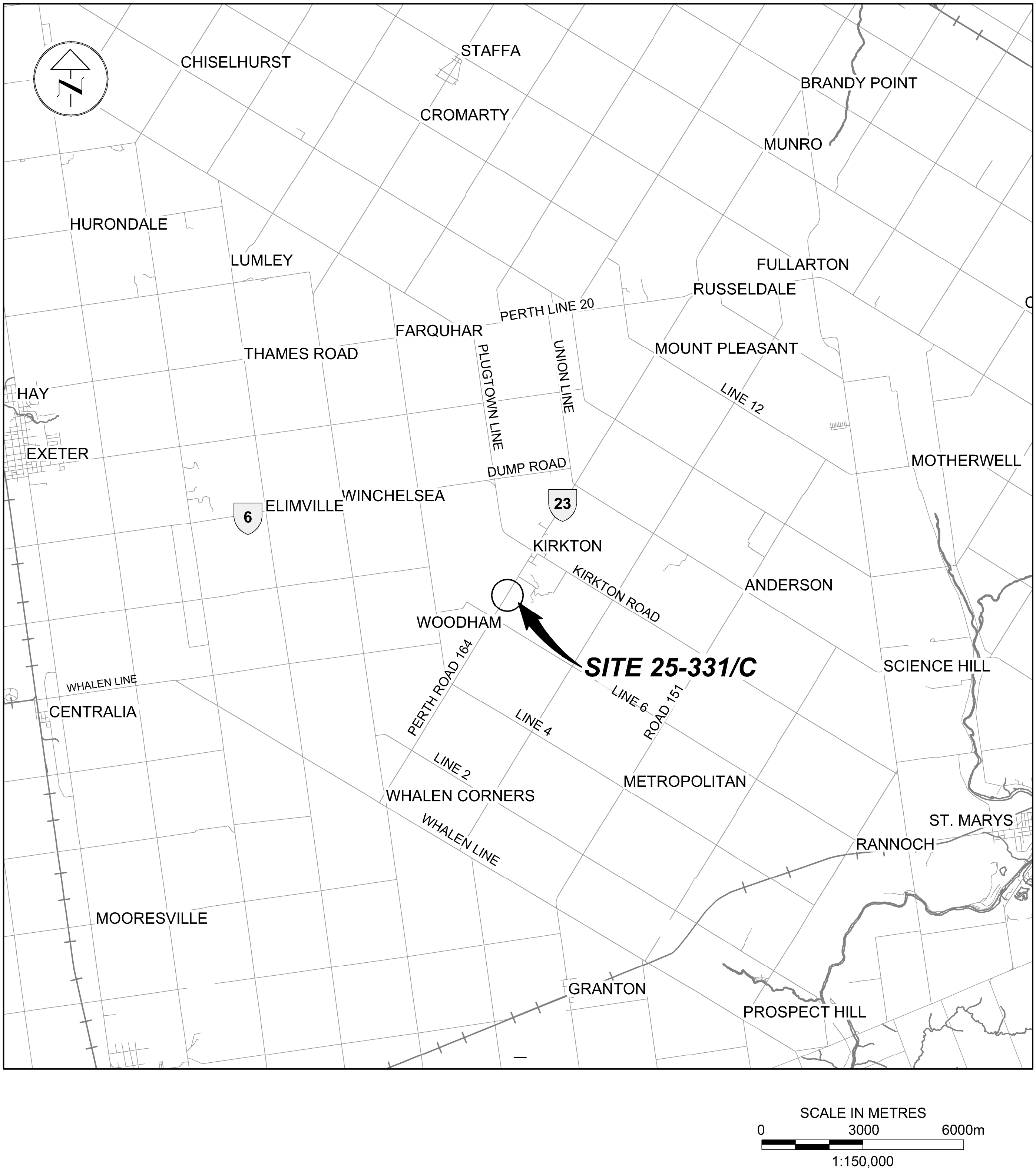
PROJECT 12-1132-0163
W.P. 3040-11-00
LOCATION N 4797715.3 , E 400337.4
ORIGINATED BY BT
DIST ____ HWY 23
BOREHOLE TYPE POWER AUGER, HOLLOW STEM
COMPILED BY AMG/LMK
DATUM GEODETIC
DATE July 16, 2013
CHECKED BY ____

ELEV DEPTH	DESCRIPTION	SAMPLES NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR	SA	SI	CL
299.31	GROUND SURFACE								
0.00	TOPSOIL, silty Brown				299				
298.91 0.40	CLAYEY SILT, trace sand, trace topsoil Firm Brown and grey	1	SS	5					
297.94 1.37	CLAYEY SILT, trace sand Firm to very stiff Grey	2	SS	7	298				
		3	SS	16	297	0	0	63	37
296.26 3.05	SANDY SILT TILL, some clay, trace to some gravel Compact to very dense Grey	4	SS	16	296				
		5	SS	20	295	10	28	41	21
		6	SS	26	294				
		7	SS	73	293				
		8	SS	102	292				
291.23 8.08	END OF BOREHOLE Borehole dry during drilling on July 16, 2013.								



UNIT WEIGHT γ kN/m³

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



REFERENCE

PLAN BASED ON CANMAP STREETFILES V.2008.5.

NOTE

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

PROJECT

WASHBURN DRAIN, SITE 25-331/C
STATION 16+510, HIGHWAY 23
GWP 3040-11-00

TITLE

KEY PLAN

Golder Associates
LONDON, ONTARIO

PROJECT No.		12-1132-0163	FILE No. 1211320163-2000-F04001
			SCALE AS SHOWN REV. 0
CADD	AMG/LMK	May 07/14	**FIGURE 1**
CHECK			

Drawing file: 1211320163-2000-F04001.dwg Oct 10, 2014 - 8:19am

MINISTRY OF TRANSPORTATION, ONTARIO

CONT No.
WP No. 3040-11-00

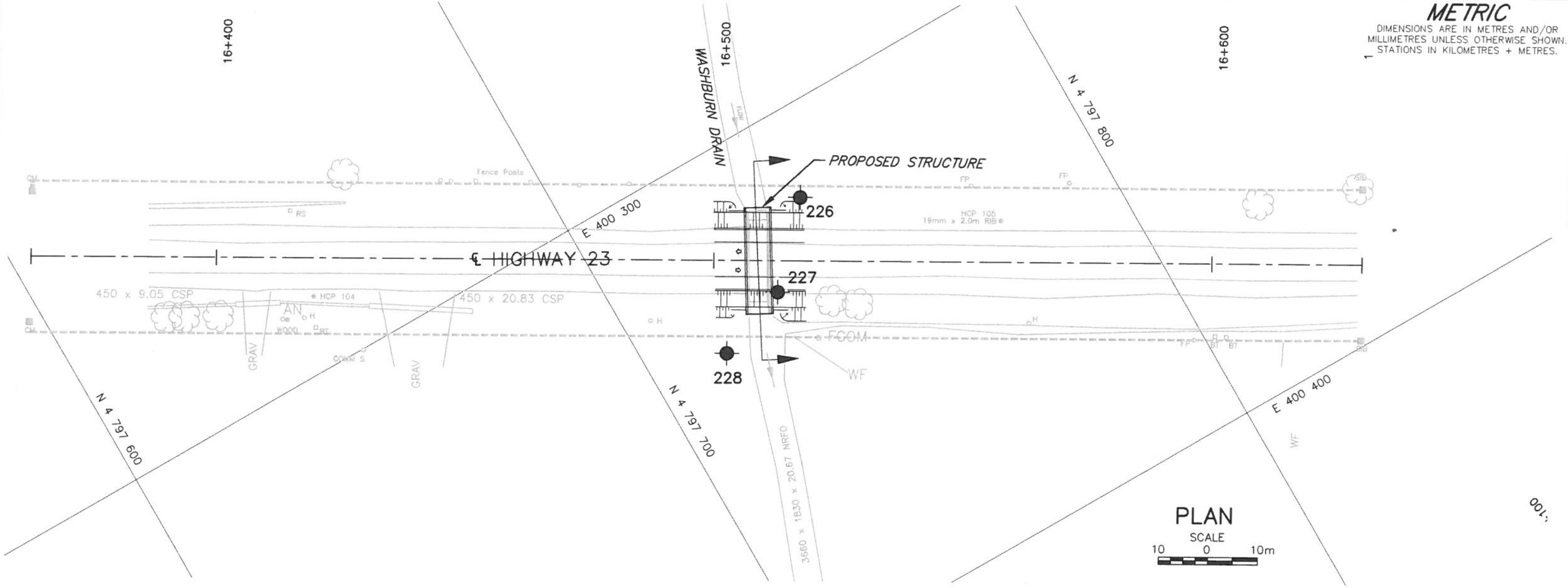
SHEET

WASHBURN DRAIN
STATION 16+510, HIGHWAY 23

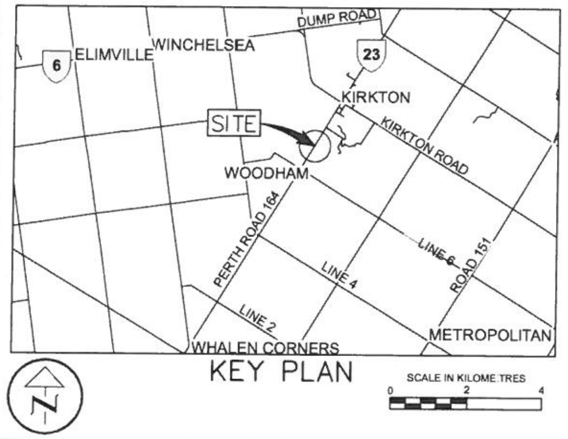
STRUCTURE REPLACEMENTS AND REHABILITATION
BOREHOLE LOCATIONS AND SOIL STRATA

Golder Associates Ltd.
LONDON, ONTARIO, CANADA

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



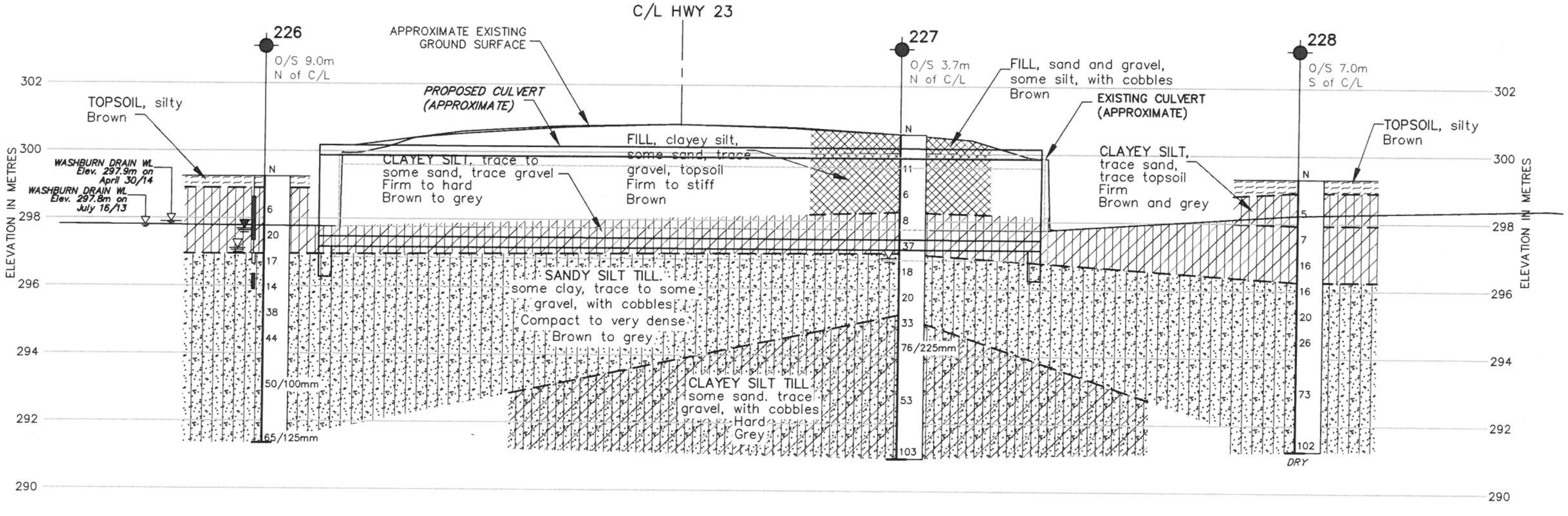
PLAN
SCALE
10 0 10m



KEY PLAN

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 measured on July 25, 2014
- ▽ WL encountered during drilling
- *DRY* Water level not established



PROFILE ALONG ℄ CULVERT
HORIZONTAL SCALE 2 0 2 m
VERTICAL SCALE 2 0 2 m

No.	ELEVATION	CO-ORDINATES (MTM NAD83 ZONE 11)	
		NORTHING	EASTING
226	299.24	4 797 743.6	400 317.5
227	300.56	4 797 730.2	400 331.8
228	299.31	4 797 715.3	400 337.4

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.

D. J. PROUT
Oct.10/14

F. J. HEFFERNAN
Oct 10/14

NO.	DATE	BY	REVISION

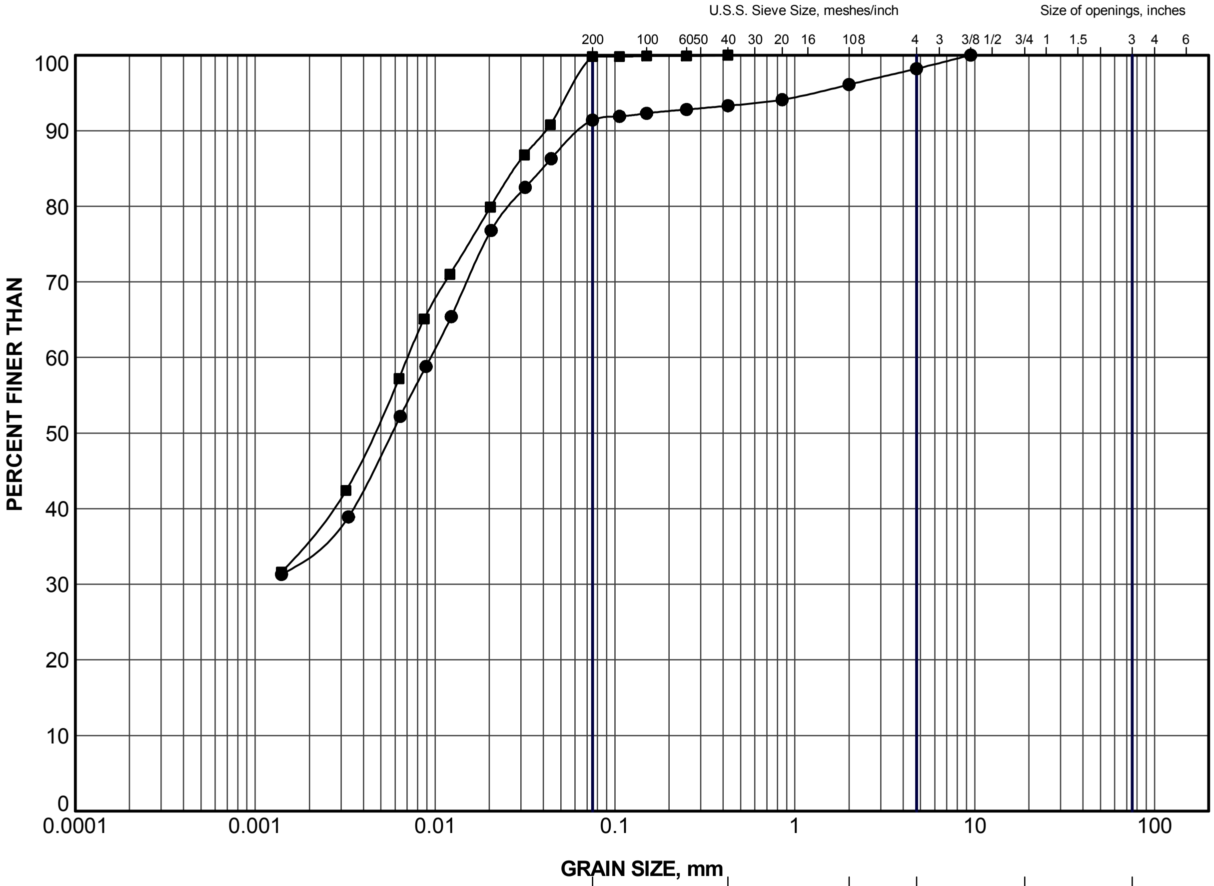
Geocres No. 40P6-28

HWY. 23		PROJECT NO. 12-1132-0163	DIST.
SUBM'D. BT	CHKD. DUP	DATE: Sept. 24/14	SITE: 25-331/C
DRAWN: WDF	CHKD. AMH	APPD. FJH	DWG. 1

PLOT DATE: October 10, 2014
FILENAME: N:\active\2012\1132 - Geo\1132-0100\12-1132-0163 STANTEC-FDNS-MEGA CULVERTS-3011-E-0041\Drafting\AutoCAD files\1211320163-2000-D04001.dwg

APPENDIX A

Laboratory Test Data



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	227	4	297.3
■	228	3	296.8

PROJECT

WASHBURN DRAIN, SITE 25-331/C
STATION 16+510, HIGHWAY 23
GWP 3040-11-00

TITLE

GRAIN SIZE DISTRIBUTION

CLAYEY SILT

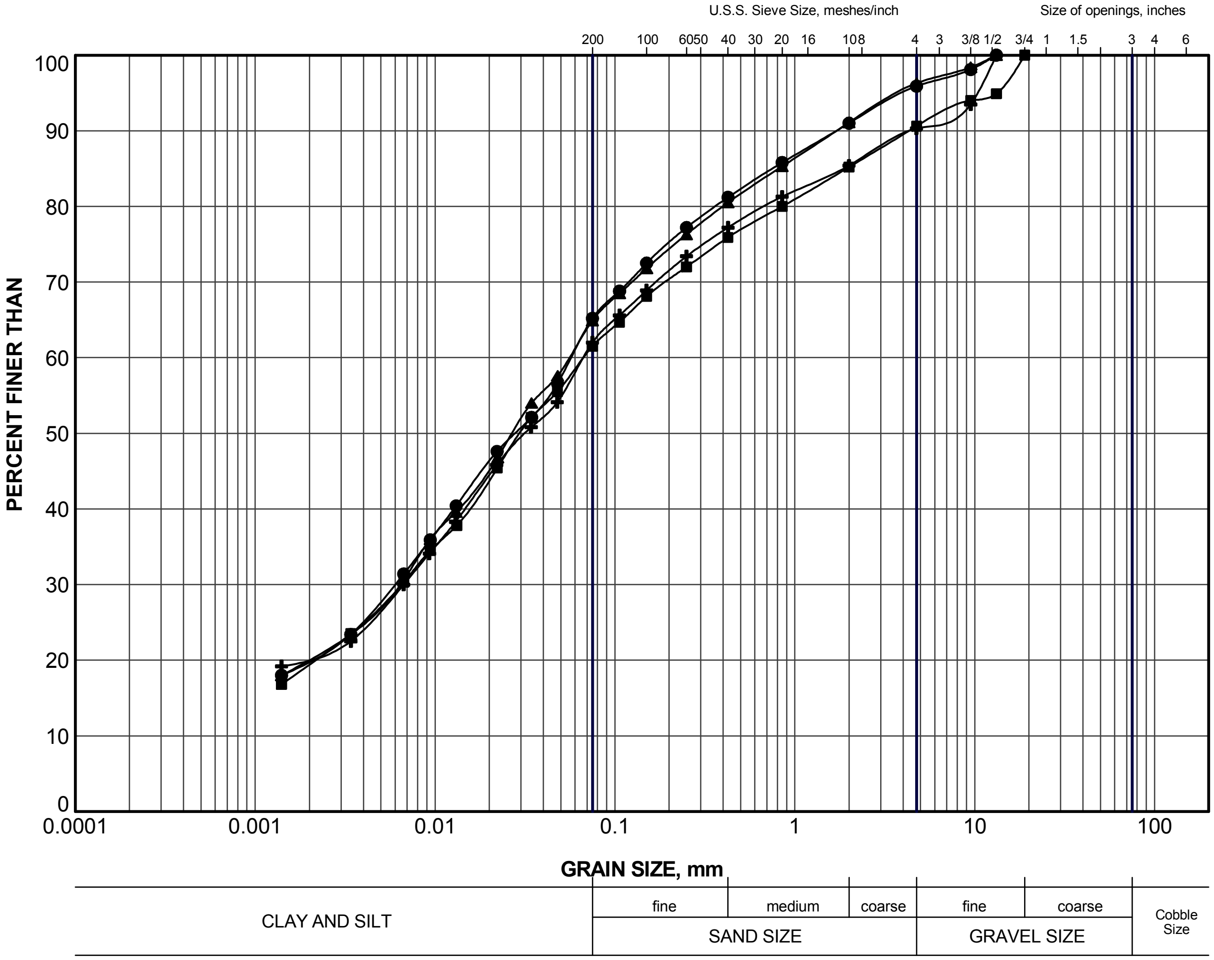
Golder Associates
LONDON, ONTARIO

PROJECT No.		12-1132-0163
DRAWN	LMK	Nov 18/13
CHECK		

FILE No. 1211320163-2000-F040A1
SCALE N/A
REV.

FIGURE A-1

LDN_MTO_GSD GLDR_LDN.GDT



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	226	3	296.7
■	226	5	295.2
▲	227	6	295.8
✚	228	5	295.3

PROJECT

WASHBURN DRAIN, SITE 25-331/C
STATION 16+510, HIGHWAY 23
GWP 3040-11-00

TITLE

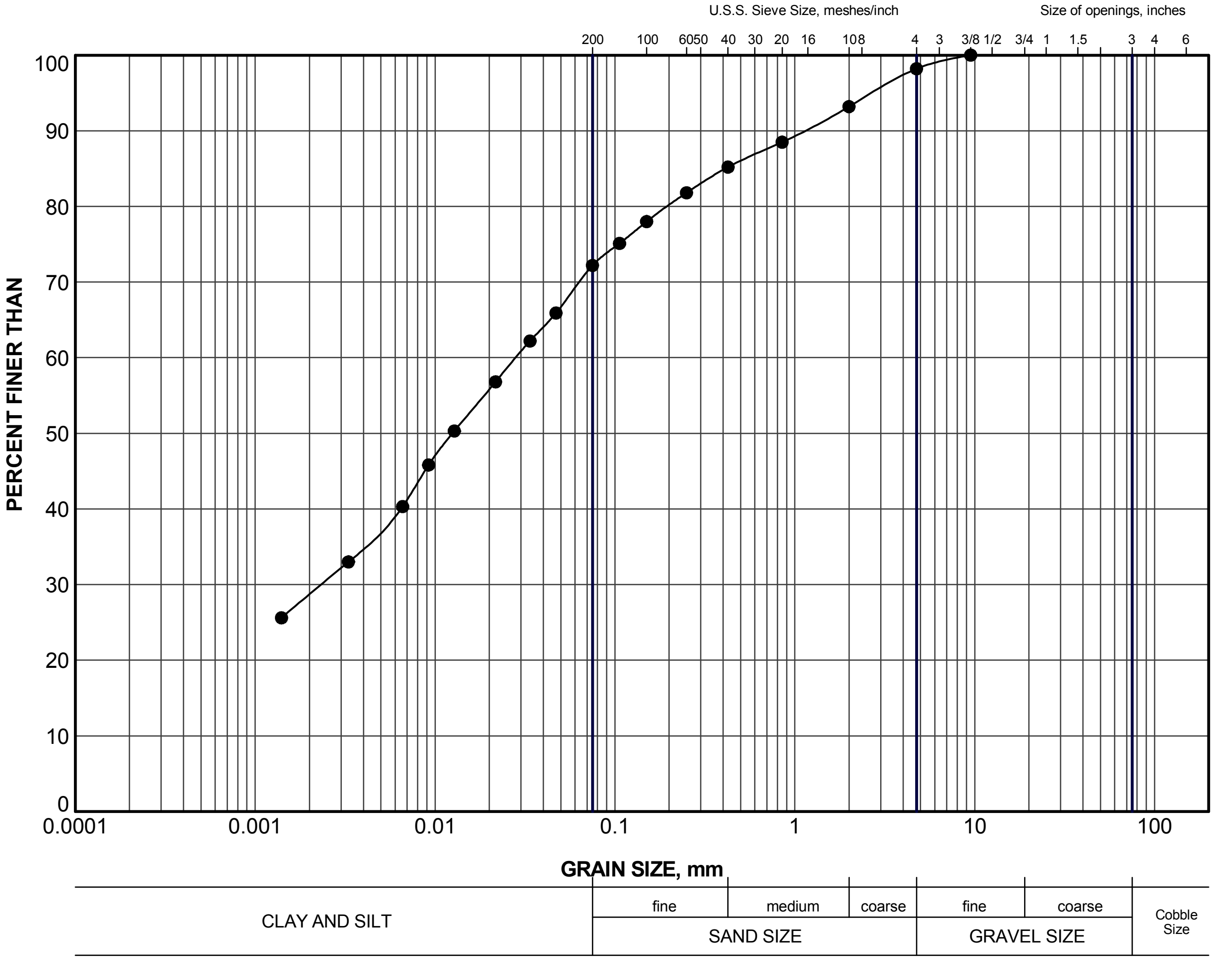
GRAIN SIZE DISTRIBUTION

SANDY SILT TILL

Golder Associates
LONDON, ONTARIO

PROJECT No.		12-1132-0163	FILE No. 1211320163-2000-F040A2
			SCALE N/A / REV.
DRAWN	LMK	Nov 18/13	**FIGURE A-2**
CHECK			

LDN_MTO_GSD GLDR_LDN.GDT



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	227	9	292.7

PROJECT

WASHBURN DRAIN, SITE 25-331/C
STATION 16+510, HIGHWAY 23
GWP 3040-11-00

TITLE

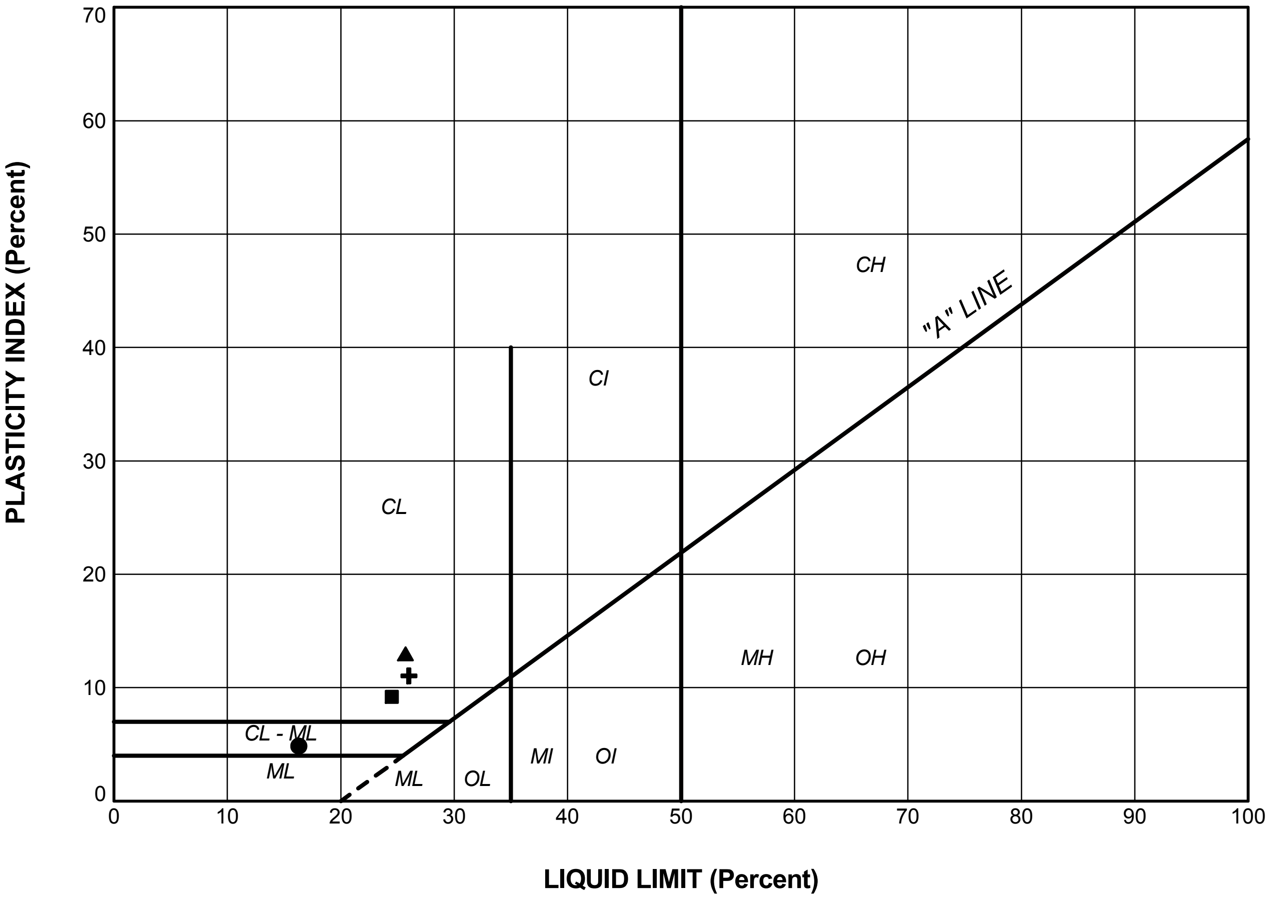
GRAIN SIZE DISTRIBUTION

CLAYEY SILT TILL

Golder Associates
LONDON, ONTARIO

PROJECT No.	12-1132-0163	FILE No.	1211320163-2000-F040A3
		SCALE N/A	REV.
DRAWN	LMK	Nov 18/13	**FIGURE A-3**
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
SANDY SILT TILL					
●	226	3	16.3	11.5	4.9
CLAYEY SILT					
■	227	4	24.5	15.3	9.2
✚	228	3	26.0	15.0	11.1
CLAYEY SILT TILL					
▲	227	9	25.7	12.7	13.0

PROJECT

WASHBURN DRAIN, SITE 25-331/C
STATION 16+510, HIGHWAY 23
GWP 3040-11-00

TITLE

PLASTICITY CHART

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PROJECT No.		12-1132-0163	FILE No. 1211320163-2000-F040A4	
			SCALE N/A	REV.
DRAWN	LMK	Nov 18/13	**FIGURE A-4**	
CHECK				

LDN_MTO_PI GLDR_LDN.GDT

APPENDIX B

Site Photographs

Photograph 1: West elevation (inlet) of Culvert Site 25-331/C.

Photograph 2: East elevation (outlet) of Culvert Site 25-331/C.

Photograph 3: Highway 23 looking north from east shoulder at Culvert Site 25-331/C.

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