



April 2017

FOUNDATION INVESTIGATION AND DESIGN REPORT

**Culvert Replacement
Site No. 35-594/C, Highway 9
Contract 4 Structure Replacements and
Rehabilitation
GWP 3042-11-00
Ministry of Transportation, Ontario - West Region**

Submitted to:

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REPORT



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LIST OF SYMBOLS

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PART A

FOUNDATION INVESTIGATION REPORT

CULVERT REPLACEMENT, DRAIN 1 CULVERT

SITE NO. 35-594/C, HIGHWAY 9

CONTRACT 4 STRUCTURE REPLACEMENTS AND REHABILITATION

GWP 3042-11-00

MINISTRY OF TRANSPORTATION, ONTARIO - 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 3042-11-00. The project involves the detailed design of the replacement and rehabilitation of several structures along multiple highways in southern Ontario. This report addresses the proposed replacement of the Drain 1 culvert (Site 35-594/C) at Station 20+011 on Highway 9 in the Geographic Township of Minto in Wellington County, Ontario.

The purpose of the foundation investigation is to explore the subsurface conditions at the location of the proposed culvert 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 and change order 12-1132-0163-4000-CO3 dated September 16, 2016. The work was carried out in accordance with our Quality Control Plan for Foundation Engineering dated March 26, 2013.

2.0 SITE DESCRIPTION

For the purposes of this report, Highway 9 has been assumed to be oriented east-west and Drain 1 to be flowing beneath Highway 9 from north to south. The culvert is located at about Station 20+011 on Highway 9 approximately 650 metres east of Ayton Road in Wellington County, Ontario. The Community of Harriston is located approximately 3.4 kilometres to the east of the site. The approximate location of the culvert is shown on the Key Plan, Figure 1. Photographs of the culvert are provided in Appendix B.

This section of Highway 9 is currently a two lane, undivided highway with gravel shoulders. Based on the information provided, the existing open footing, non-rigid frame culvert was previously extended to the north and south. A concrete floor slab was constructed and miscellaneous concrete repairs were carried out circa 2001. The existing culvert has an overall length of about 18.1 metres.

Dimensions (m)	Obvert Elevation (m)		Construction
	Lt ¹	Rt ¹	
3.05 x 2.24 x 18.1	375.80	375.79	Concrete Open Footing

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

Gabion retaining walls are located adjacent to the culvert in each of the four quadrants. There is some slight bulging in the retaining walls and some longitudinal and transverse cracking of the asphalt riding surface. The banks of Drain 1 and the embankments along Highway 9 adjacent to the culvert are grass covered.



2.1 Site Geology

The site lies within the Teeswater Drumlin Field physiographic region¹. The till is characterized as loamy in texture, moderately compact, highly calcareous and pale brown or yellowish brown in colour. Valleys are typically filled with broad terraces of sand and gravel. The quaternary geology mapping indicates that the surficial soils in the area of the site generally consist of glaciofluvial outwash deposits of sand with minor gravel.² The underlying bedrock surface is typically found at about elevation 358 metres and the overburden thickness is typically some 16 to 19 metres. The rock formation is mapped and described as dolomite of the Bass Islands Formation of Upper Silurian age³.

3.0 INVESTIGATION PROCEDURES

The field work for the investigation was carried out on September 27, 2016, during which time three boreholes were drilled at the approximate locations shown on the Borehole Location Plan, Drawing 1.

The boreholes were drilled using a D50T track mounted drill rig supplied and operated by a specialist drilling contractor. Samples of the overburden were typically obtained at intervals of 0.75 metres to 6 metres depth and at 1.5 metre intervals thereafter using 50 millimetre outside diameter split spoon sampling equipment in accordance with the Standard Penetration Test (SPT) procedures (ASTM D1586). The results of the SPT testing, as presented on the Record of Borehole sheets, Drawing 1 and in Section 4.0 of this report, are unmodified (not standardized for hammer efficiency, borehole diameter, rod length, etc.). The samplers used in the investigation limit the maximum particle size that can be sampled and tested to about 40 millimetres; therefore, particles or objects that may exist within the soils that are larger than this dimension will not be sampled or represented in the grain size distributions. Larger particle sizes, including cobbles and boulders, were inferred in the fill and native deposits as discussed in the text of this report.

Groundwater conditions in the boreholes were observed throughout the drilling operations and a piezometer was installed in BH-603 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 a member of our staff who located the boreholes in the field, obtained utility locates, 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, grain size distribution analyses and an Atterberg limits determinations, 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 at the borehole locations 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.

¹ Chapman, L.J., and Putnam, D.F., 1984: Physiography of Southern Ontario; Ontario Geological Survey, Special Volume 2, 270p.

² Quaternary Geology of the Palmerston Area, Southern Ontario; Ontario Div. Mines, Prelim. Map. P.1185, Geol. Ser., Scale 1:50,000. Geology 1972, 1973.

³ Ontario Division of Mines, Map 2254, Paleozoic Geology of Southern Ontario Showing Bedrock Industrial Mineral Producers, Scale 1:1,013,760.



Borehole	Location (m)		Ground Surface Elevation (m)	Borehole Depth (m)
	Northing	Easting		
BH-601	4 866 684	191 903	376.47	11.13
BH-602	4 866 686	191 890	376.48	11.13
BH-603	4 866 665	191 904	374.49	8.08

4.0 SUBSURFACE CONDITIONS

4.1 Site Stratigraphy

The subsurface soil and groundwater conditions encountered in the boreholes, together with the results of the in situ testing and 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 or surficial topsoil and embankment fill materials overlying native granular soils, 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

The existing pavement structure was encountered in BH-601. The pavement consisted of about 70 millimetres of asphalt and 630 millimetres of crushed granular base. About 430 millimetres of granular base was encountered at the shoulder surface in BH-602.

Surficial topsoil, about 430 millimetres thick, was encountered at ground surface in BH-603. In addition, a layer of buried topsoil, about 0.8 metres thick, was encountered at about elevation 374.3 metres in BH-601. The buried topsoil had an N value, as determined in the standard penetration testing, of 8 blows per 0.3 metres. 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.

Loose to compact silty sand to sandy silt fill, was encountered in BH-601 at elevation 375.8 metres and in BH-602 at elevation 376.1 metres. The fill was 1.4 and 2.5 metres thick in BH-601 and BH-602, respectively. The fill had N values, ranging from 5 to 16 blows per 0.3 metres. A sample of the fill from BH-601 had a water content of about 19 per cent. Cobbles were inferred in the fill in BH-602 and wood pieces and topsoil were noted. A grain size distribution curve for a sample of the fill from BH-601 is presented on Figure A-1 in Appendix A.



A layer of stiff clayey silt was encountered at elevation 373.6 metres in BH-601. The clayey silt was about 0.8 metres thick and had an N value of 9 blows per 0.3 metres. Topsoil layers were noted in the clayey silt.

Layers of compact to very dense silty sand and gravel were encountered in BH-601 at elevation 372.8 metres, in BH-602 at elevation 373.6 metres and in BH-603 at elevation 373.4 metres. The silty sand and gravel layers ranged in thickness from 1.5 to 3.4 metres. The silty sand and gravel had N values of 17 to 60 blows per 0.3 metres. Samples of the silty sand and gravel had water contents of about 8 and 10 per cent. Cobbles and boulders should be expected in the silty sand and gravel deposits. Grain size distribution curves for samples of the silty sand and gravel from BH-601 and BH-603 are presented on Figure A-2.

Compact sandy silt was encountered at elevation 372.8 metres in BH-602 within the silty sand and gravel and beneath the topsoil in BH-603 at elevation 374.1 metres. The sandy silt layers in BH-602 and BH-603 were about 0.8 and 0.6 metres thick, respectively. The sandy silt had N values of 16 and 22 blows per 0.3 metres with a water content of about 10 per cent. A grain size distribution curve for a sample of the sandy silt from BH-602 is presented on Figure A-3.

Compact to very dense sandy silt glacial till was encountered beneath the silty sand and gravel in all of the boreholes between elevation 370.1 and 371.3 metres. BH-602 was terminated in the sandy silt till after exploring it for about 5.2 metres. The sandy silt till was about 4.9 and 2.6 metres thick in BH-601 and BH-603, respectively. The sandy silt till had N values of 22 to over 100 blows per 0.3 metres and water contents of about 8 and 9 per cent. Cobbles were noted and boulders should be expected in the sandy silt till. Grain size distribution curves for samples of the sandy silt till are presented on Figure A-4.

Hard clayey silt glacial till was encountered beneath the sandy silt till in BH-601 at elevation 366.4 metres. This borehole was terminated in the clayey silt till after exploring it for about 1.1 metres. The clayey silt till had an N value of 31 blows per 0.3 metres and a water content of about 16 per cent. The clayey silt till had plastic and liquid limits of about 13 and 21 per cent, respectively, based on a single Atterberg limits determination. These data are shown on Figure A-6. Cobbles and boulders should be expected in the clayey silt glacial till. A grain size distribution curve for a sample of the clayey silt glacial till is presented on Figure A-5.

Beneath the sandy silt till, a layer of very dense silt was encountered in BH-603 at elevation 367.5 metres. BH-603 was terminated in the silt after exploring it for about 1.1 metres. The silt had an N value of 64 blows per 0.3 metres.

4.3 Groundwater Conditions

Groundwater conditions were observed during and on completion of drilling and a groundwater observation piezometer was installed in BH-603. The installation details are provided on the corresponding Record of Borehole sheet following the text of this report. Groundwater was encountered in the boreholes during drilling between depths of 1.5 and 3.7 metres or between elevation 372.8 and 373.6 metres. The water level in Drain 1 was measured at elevation 373.56 metres on September 27, 2016. The General Arrangement Drawing indicated an approximate drain water level of 373.66 metres on November 2, 2015. On October 25, 2016 and January 5, 2017, the water level in the piezometer installed in BH-603 was about 0.7 and 0.6 metres below ground surface or at about elevation 373.8 and 373.9 metres, respectively. A summary of the encountered and measured groundwater levels is provided in the table below.



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Borehole	Ground Surface Elevation (m)	Encountered Groundwater Elevation (m)	Measured Groundwater Level Elevation (m)	
			October 25, 2016	January 5, 2017
BH-601	376.47	372.8	-	-
BH-602	376.48	373.6	-	-
BH-603	374.49	373.0	373.80	373.91

The above-noted encountered and measured groundwater levels are not considered to be representative of the long-term, stabilized groundwater conditions. Based on the observed groundwater levels, the surrounding topography and the changes in soil colour, the long-term groundwater level is inferred to typically be at about elevation 374 metres. The groundwater levels should be expected to fluctuate seasonally and be higher during periods of sustained precipitation or during spring snow melt conditions.

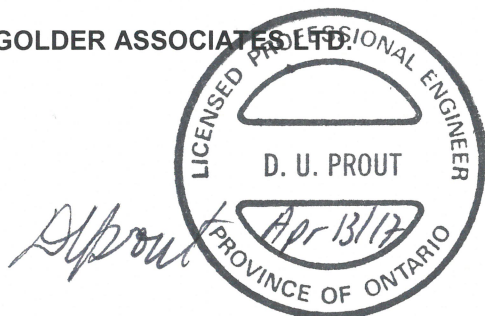


FOUNDATION INVESTIGATION AND DESIGN REPORT CULVERT REPLACEMENT, DRAIN 1, SITE NO. 35-594/C, HIGHWAY 9

5.0 MISCELLANEOUS

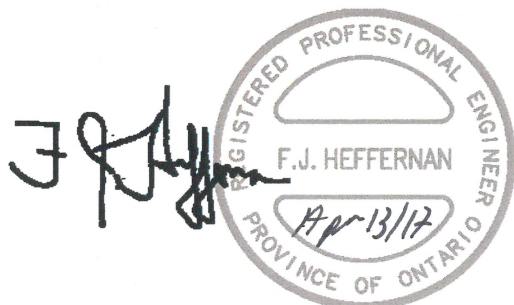
The investigation was carried out using equipment supplied and operated by London Soil Test Ltd., an Ontario Ministry of Environment and Climate Change licensed well contractor. The field operations were supervised by Mr. Daniel Hyland, E.I.T. under the direction of the Field Investigation Manager, Mr. Brett Thorner, P.Eng. The laboratory testing was carried out at Golder Associates' London laboratory under the direction of Mr. Michael Arthur. 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. Daniel Hyland, E.I.T. under the direction of the Project Engineer, Ms. Dirka U. Prout, P.Eng. The report was reviewed by Mr. Michael E. Beadle, P.Eng., an Associate with Golder Associates. Mr. Fintan J. Heffernan, P.Eng., the Designated MTO Contact and Quality Control Auditor for this assignment, conducted an independent quality review of the report.

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PART B

FOUNDATION DESIGN REPORT

CULVERT REPLACEMENT, DRAIN 1 CULVERT

SITE NO. 35-594/C, HIGHWAY 9

CONTRACT 4 STRUCTURE REPLACEMENTS AND REHABILITATION

GWP 3042-11-00

MINISTRY OF TRANSPORTATION, ONTARIO - WEST REGION



6.0 ENGINEERING RECOMMENDATIONS

This section of the report provides our recommendations on the foundation aspects of the design of the proposed culvert replacement at Site 35-594/C at Station 20+011 on Highway 9 in the Geographic Township of Minto in Wellington County, Ontario.

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

Based on the information provided, the existing structure is a concrete, non-rigid frame, open footing culvert. Based on our interpretation of the information provided, the north and south extensions are each about 4 metres in length. The original structure had a length of about 10.12 metres. The culvert has a span of about 3.05 metres and a 2.24 metre high opening with an approximate invert elevation of 373.4 metres. Gabion walls, less than 2 metres in height are present adjacent to the culvert in each quadrant.

Based on information provided by Stantec, the existing culvert will be removed and replaced with a new precast structure. Recommendations have been included for both a new open footing culvert and a new box culvert. It is understood that the proposed culvert will have approximately the same footprint as the existing culvert and a similar founding elevation.

6.1 Consequence and Site Understanding Classification

In accordance with Section 6.5 of the Canadian Highway Bridge Design Code (CHBDC version S6-14) and its Commentary, a classification of 'typical' consequence has been assumed for the proposed replacement culvert and foundation system. This consequence classification should be confirmed by Stantec and the MTO.

The degree of understanding, based on the scope of the foundation investigation and proximity of the boreholes to the culvert, is considered 'typical' as described in Clause 6.5.3.2 of the 2014 CHBDC. The appropriate Ultimate Limit States (ULS) and Serviceability Limit States (SLS) consequence factors, Ψ , geotechnical resistance factors at ULS (ϕ_{gu}) and SLS (ϕ_{gs}), respectively, from Tables 6.1 and 6.2 of the CHBDC should be used for design.

6.2 Foundations

Based on the results of the investigation, the new box culvert may be founded at or below elevation 373.4 metres in the silty sand and gravel or the stiff clayey silt. The culvert foundations may be designed using a factored geotechnical resistance at ULS of 350 kilopascals (kPa) and a geotechnical reaction at SLS of 200 kPa. The SLS value corresponds to 25 millimetres of settlement. Bedding for a precast culvert and a levelling pad should be provided as discussed in Section 6.2.3 below. In the event that an open footing culvert is constructed, the footings may be founded at or below elevation 372 metres using the same geotechnical resistances provided above.

Should unsuitable material be present at the founding level, it should be subexcavated and replaced with suitable granular fill. The fill should consist of uniformly compacted Granular A or Granular B Type II placed in maximum



300 millimetre thick loose lifts in accordance with Ontario Provincial Standard Specification (OPSS).PROV 501. The engineered fill should extend a minimum distance of 1 metre beyond the edge of the culvert and downwards and outwards at 1 horizontal to 1 vertical to intersect competent native soils.

If the footings cannot be poured or the bedding placed promptly following inspection, a 100 millimetre thick working slab of lean concrete should be provided to preserve the integrity of the founding soils.

If the underside of footing elevation or underside of levelling pad is higher than the underside of existing footing elevation, the existing footings should be fully removed and the excavations backfilled with lean concrete following inspection by qualified geotechnical personnel.

6.2.1 Frost and Scour Protection

Frost treatment, in the form of a frost taper symmetrical about the culvert centreline, should 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 1.4 metres below ground surface. Open footing culvert foundations should be provided with soil cover of a depth equivalent to the frost penetration depth or equivalent thermal substitute. The culvert should also be adequately protected against scour as noted in Section 1.9.5 of the CHBDC.

6.2.2 Resistance to Lateral Forces/Sliding Resistance

The resistance to lateral forces/sliding resistance between the base of the culvert and the bedding or native silty sand and gravel should be calculated in accordance with Section 6.10.5 of the CHBDC. Cut-off walls should be provided in accordance with Clause 1.9.5.6 of the CHBDC.

The factored horizontal geotechnical resistance, H_{ri} and H_{rs} , is calculated as follows:

$$H_{ri} = \psi \phi_{gu} (A'c'_i + V_f \tan \delta'_i) > H_f \text{ (for pre-cast elements)}$$

$$H_{rs} = \psi \phi_{gu} (A'c' + V_f \tan \phi') > H_f \text{ (for open footing culverts)}$$

Where:

ψ	=	consequence factor, given in Section 6.5.2, Table 6.1 of the CHBDC
ϕ_{gu}	=	ultimate geotechnical resistance factor, given in Section 6.9.1, Table 6.2 of the CHBDC
A'	=	effective contact area, square metres
c'_i	=	effective cohesion along the interface between the box culvert base and bedding/levelling, nil
c'	=	effective cohesion, nil
$\tan \delta'_i$	=	coefficient of friction for interface between box culvert base and bedding/levelling pad



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$\tan\phi'$	=	coefficient of friction between footings and native founding soils
V_f	=	factored vertical force, kilonewtons
H_f	=	factored horizontal load, kilonewtons

The factored horizontal resistance may be calculated using the parameters in the following table:

Structure	Interaction	Angle of Friction, δ/ϕ (degrees)	Coefficient of Friction, $\tan \delta/\phi$
Precast Box Culvert	Precast concrete on Granular A bedding/levelling pad	30	0.58
Open Footing Culvert	Cast-in-Place concrete on native sand and gravel	32	0.62

6.2.3 Bedding, Backfill and Cover

Backfill for the culvert shall be placed in accordance with OPSS.PROV 501 and OPSS 422. Backfill for the culvert should consist of free-draining, non-frost susceptible granular materials such as OPSS Granular B Type III, Type II or Granular A placed in 0.3 metre thick loose lifts and uniformly compacted. 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 as equal as possible on both sides of the culvert during all stages of backfill placement. The height of the backfill on each side of the culvert should differ by no more than 500 millimetres at any time.

The excavations for this culvert should have a clearance width that exceeds the width of the culvert by at least 1.0 m on each side to allow for good workmanship and effective compaction of the fill. Bedding for a precast box culvert is to be placed on properly prepared native competent materials or approved compacted granular materials and should be at least 300 millimetres thick. At no time should the culvert be constructed on frozen materials. Granular A is considered to be suitable for use as bedding for a precast box culvert. The levelling course can consist of a 75 millimetre thick layer of Granular A or materials meeting the gradation requirements for fine concrete aggregates.

6.2.4 Other Design Considerations

The fill height above the culvert roof will be less than 4.5 metres and no grade raise or widening is planned. The foundation materials consist of granular deposits and low compressibility glacial till deposits. Therefore, differential settlement along the length of this culvert is expected to be minor and, thus, camber is not required.



If the results of hydraulic analysis indicate that there will be significant difference in hydraulic head between the inlet and outlet, then the culvert inlet should be provided with a headwall, clay seal or other seepage control measure. The design of the replacement culvert should include a cut-off wall at the inlet in accordance with Section 1.9.5.6 of the CHDBC.

Erosion protection for the culvert backfill should be provided to protect the roadway, embankments and culvert, 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. Temporary erosion protection and sediment control measures should be implemented in accordance with OPSS 805. 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 together with diversion of any flows in the drain to mitigate migration of fine soil particles.

6.3 Excavations and Groundwater Control

Excavations will extend through the existing pavement structure, fill and topsoil to the underlying native clayey silt, sandy silt and silty sand and gravel. Cobbles and boulders should be expected in the fill and silty sand and gravel. Seepage from the sandy silt and silty sand and gravel deposits are expected to be such that groundwater control can probably be achieved using properly constructed and filtered sumps in the base of the excavation. 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.

Temporary open cut slopes should be maintained no steeper than 1 horizontal to 1 vertical. Localized sloughing and ground movements should be expected in the fill. 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 soil and the native granular deposits would be classified a Type 2 soil above the groundwater level and as Type 4 soils below the groundwater level.

6.4 Retaining Walls

Based on the information provided to Golder, it is understood that the gabion walls in each quadrant will require reconstruction in conjunction with the culvert replacement. It is understood that the replacement retaining walls would be less than 2 metres in height. From a geotechnical perspective, gabion walls, precast concrete block walls, armour stone walls, concrete cantilever walls, concrete gravity walls, including precast toe walls, and retained soil system walls (RSS) are suitable alternatives. Considering wall heights of less than 2 metres, gabion walls, precast concrete block walls, armour stone walls and precast concrete toe walls are probably more economical than RSS walls and concrete cantilever or other concrete gravity walls. Gabion walls, precast concrete block walls, armour stone walls and RSS walls need not be founded at the frost depth and these wall types are most tolerant of movement. The structural designers are contemplating use of a proprietary Lock-Block system, which uses precast modular blocks stacked using an interlocking shear key to form an unreinforced gravity wall.

The retaining walls can be supported on the compact to very dense silty sand and gravel at or below elevation 373.4 metres. A factored geotechnical resistance of 150 kPa at ULS and a geotechnical reaction of 100 kPa at SLS may be used for design of the retaining walls. The SLS value corresponds to 25 millimetres of total settlement.



6.4.1 Frost Protection and Embedment

The frost depth applicable to this site is 1.4 metres. Gabion walls, precast concrete block walls, armour stone walls and RSS walls do not require an embedment depth equivalent to the frost depth provided they are founded on granular pads with a compacted thickness of at least 300 millimetres. In addition, the retaining walls should have sufficient embedment to provide stability and adequate protection against scour and erosion. Proprietary systems, such as Lock-Block, may have additional foundation preparation requirements and should be designed and constructed in accordance with the supplier's requirements.

6.4.2 Lateral Resistance

The resistance to lateral forces/sliding resistance between the underside of the retaining walls and levelling pads or subgrade soil, as applicable, should be calculated in accordance with Section 6.10.5 of the CHBDC. In addition, the retaining walls should be checked for overturning. The equations and soil parameters provided in Section 6.2.2 should be used for the interaction between the retaining walls and the founding soil. The passive resistance of the portion of the retaining walls below grade should be neglected.

6.4.3 Other Design Considerations

The retaining 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. For gabion and armour stone walls, an approved, non-woven geotextile should be placed at the rear of the walls in order to minimize clogging and or loss of fines through the stone. Gabion walls would need to be designed and constructed in accordance with OPSS.PROV512. Armour stones should be properly chinked and fitted.

6.5 Liquefaction Potential and Seismic Analysis

6.5.1 Seismic Parameters

For the purposes of this project, Site Class D is appropriate based on the results of the investigation. Seismic performance should be calculated in accordance with Section 4.4.3 of the CHBDC (version S6-14).

The importance category of the replacement culvert is "other" based on the CHBDC. The corresponding Seismic Category for the structure is 1 based on Table 4.10 of the CHBDC. Structures in Seismic Category 1 need not be analysed for seismic loads. However, the minimum requirements as outlined in CHBDC Clause 4.4.5.1 must be followed. 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.



6.5.2 Seismic Hazard Assessment

A preliminary screening of the soil stratigraphy was conducted using the procedure outlined in the Federal Highway Administration recommended procedures⁴ and Canadian Foundation Engineering Manual (CFEM). The potential for liquefaction occurring at this site is very low due to historically low seismicity in this area and founding soils not susceptible to liquefaction. Therefore, a detailed evaluation of the liquefaction potential of the foundation soils is not considered warranted.

6.6 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, the nature of the soil behind the backfill, the magnitude of surcharge including construction loadings, the freedom of lateral movement of the structure and the drainage conditions behind the walls.

The following recommendations are made concerning the design of the walls in accordance with the CHBDC (version S6-14). 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 coefficients of lateral earth pressure must be adjusted to account for the slope.

- Backfill should be placed in accordance with OPSD 3121.1500 and Sections 6.2.3 and 6.4.3 above. Where applicable, wall drains are to be incorporated into the design in accordance with OPSD 3190.1000.
- 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 1.4 metres behind the culvert walls (Case (a) from commentary on CHBDC Figure C6.20).
- For Case (a), the restrained case, the pressures are based on the existing embankment fill materials; however, since frost tapers will be provided, the pressures will be based on the materials used to construct the tapers. The following parameters (unfactored) may be used:

	<u>GRANULAR A</u>	<u>GRANULAR B TYPE II</u>	<u>GRANULAR B TYPE III</u>
Soil unit weight:	22 kN/m ³	21 kN/m ³	21 kN/m ³
Coefficients of lateral earth pressure:			
'At rest' or restrained, K_0	0.43	0.47	0.43

⁴ 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 structures, such as typically the case for retaining walls), active earth pressures may be used in the geotechnical design of the structure. The granular fill should be placed in a wedge 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:

	<u>GRANULAR A</u>	<u>GRANULAR B</u> <u>TYPE II</u>	<u>GRANULAR B</u> <u>TYPE III</u>
Fill unit weight:	22 kN/m ³	21 kN/m ³	21 kN/m ³
Coefficients of lateral earth pressure:			
'active' or unrestrained, K_a	0.27	0.31	0.27
'passive', K_p	3.7	3.2	3.7

6.7 Temporary Roadway Protection

It is understood that temporary roadway protection will be required should a single lane of traffic need to be maintained on Highway 9 at the culvert location during construction. Temporary support systems could consist of cantilevered soldier piles and lagging or steel sheet piles. Installation of steel sheets into the silty sand and gravel and glacial tills will be difficult. Difficulties achieving the design depth and alignment should be expected given the density of the soil and the presence of cobbles and boulders.

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/m²; 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



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)	Bulk Unit Weight γ (kN/m ³)	Effective Unit Weight γ' (kN/m ³)
	Active, K_a	At Rest, K_o	Passive, K_p			
Fill	0.38	0.55	Nil	-	19.0	9.0
Clayey Silt	0.36	0.53	2.8	28	19.0	9.0
Sandy Silt	0.33	0.50	3.0	30	20.0	10.0
Sand and Gravel	0.31	0.47	3.3	32	21.0	11.0
Sandy Silt Till	0.31	0.47	3.3	32	22.0	12.0
Clayey Silt Till	0.31	0.47	3.3	32	21.0	11.0

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 adjusted accordingly.

6.8 Construction Considerations

Care should be taken during construction to avoid disturbance of the subgrades prior to constructing foundations or placing bedding. All existing fill and any topsoil, organics, frozen, soft and/or loose soils should be stripped from the proposed founding areas prior to placement of the bedding materials or working mat. 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 base should be inspected by the QVE.

A Non-standard Special Provision (NSSP) or Notice to the Contractor should be added to the Contract Documents to advise the Contractor of the potential for cobbles and boulders in the fill, silty sand and gravel and glacial tills. Example text for an NSSP alerting the Contractor to the presence of cobbles and boulders, and for the potential need for a working mat, have been included in Appendix C.



FOUNDATION INVESTIGATION AND DESIGN REPORT CULVERT REPLACEMENT, DRAIN 1, SITE NO. 35-594/C, HIGHWAY 9

7.0 MISCELLANEOUS

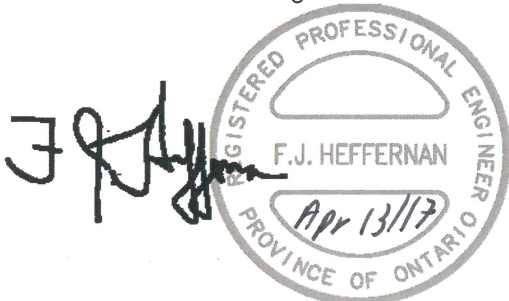
This section of the report was prepared by Mr. Daniel W. Hyland, E.I.T. under the direction of the Project Engineer Ms. Dirka U. Prout, P.Eng. The report was reviewed by Mr. Michael E. Beadle, P.Eng. Mr. Fintan J. Heffernan, P.Eng., the Designated MTO Contact and Quality Control Auditor for this assignment conducted an independent quality review of the report.

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MTO Designated Contact

DH/DUP/MEB/FJH/CR

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n:\active\2012\1132 - geo\1132-0100\12-1132-0163 stantec-fdns-mega culverts-3011-e-0041\ph 4000-gwp 3042-11-00\rpts\r06 35-594c drain 1\1211320163-4000-r06 apr 12 17 (final) part a&b fdns replace clvrt 35-594c.docx



LIST OF SYMBOLS

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

I. GENERAL

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

II. STRESS AND STRAIN

γ	shear strain
Δ	change in, e.g. in stress: $\Delta \sigma$
ε	linear strain
ε_v	volumetric strain
η	coefficient of viscosity
ν	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 or LL	liquid limit
w_p or PL	plastic limit
I_p or PI	plasticity index = $(w_l - w_p)$
w_s	shrinkage limit
I_L	liquidity index = $(w - w_p) / I_p$
I_C	consistency index = $(w_l - w) / I_p$
e_{max}	void ratio in loosest state
e_{min}	void ratio in densest state
I_D	density index = $(e_{max} - e) / (e_{max} - e_{min})$ (formerly relative density)

(b) Hydraulic Properties

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

(c) Consolidation (one-dimensional)

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

(d) Shear Strength

τ_p, τ_r	peak and residual shear strength
ϕ'	effective angle of internal friction
δ	angle of interface friction
μ	coefficient of friction = $\tan \delta$
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

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

Notes: 1
2

$$\tau = c' + \sigma' \tan \phi'$$

$$\text{shear strength} = (\text{compressive strength})/2$$



LIST OF ABBREVIATIONS

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

I. SAMPLE TYPE

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

II. PENETRATION RESISTANCE

Standard Penetration Resistance (SPT), N:

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

Dynamic Cone Penetration Resistance; N_d :

The number of blows by a 63.5 kg (140 lb.) hammer dropped 760 mm (30 in.) to drive uncased a 50 mm (2 in.) diameter, 60° 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 cm² pushed through ground at a penetration rate of 2 cm/s. Measurements of tip resistance (Q_t), porewater pressure (PWP) and friction along a sleeve are recorded electronically at 25 mm penetration intervals.

III. SOIL DESCRIPTION

(a) Non-Cohesive (Cohesionless) Soils

Density Index	N
Relative Density	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.

V. MINOR SOIL CONSTITUENTS

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

RECORD OF BOREHOLE No BH-601

1 OF 1

METRIC

PROJECT 12-1132-0163

W.P. 3042-11-00

LOCATION N 4866683.7, E 191903.2

ORIGINATED BY DH

DIST _____ HWY 9

BOREHOLE TYPE POWER AUGER, HOLLOW STEM

COMPILED BY ZJB/LMK

DATUM GEODETIC

DATE September 27, 2016

CHECKED BY

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _P	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa										WATER CONTENT (%)		
								20 40 60 80 100										10 20 30		
376.47	PAVEMENT SURFACE																			
0.07	ASPHALT																			
375.77	FILL, sand and crushed gravel Brown																			
0.70	FILL, silty sand, trace to some clay, trace gravel, with topsoil layers Loose Brown		1	SS	5											3 44 44 9				
374.34			2	SS	5															
2.13	TOPSOIL, silty Loose Brown		3	SS	8															
373.57			4	SS	9															
2.90	CLAYEY SILT, some sand, with topsoil layers Stiff Brown and grey		5	SS	51															
372.81			6	SS	56											29 48 19 4				
3.66	SILTY SAND AND GRAVEL Very dense Brown		7	SS	40															
371.29			8	SS	26											7 40 44 9				
5.18	SANDY SILT TILL, trace to some clay, trace to some gravel, with cobbles Compact to very dense Brown		9	SS	100/ 76mm															
			10	SS	100/ 279mm															
366.41																				
10.06	CLAYEY SILT TILL, some sand, trace gravel, with silt seams Hard Brown		11	SS	31											0 10 61 29				
365.34																				
11.13	END OF BOREHOLE																			
	Groundwater encountered at about elev. 372.8m during drilling on September 27, 2016.																			

RECORD OF BOREHOLE No BH-602

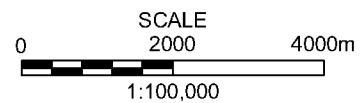
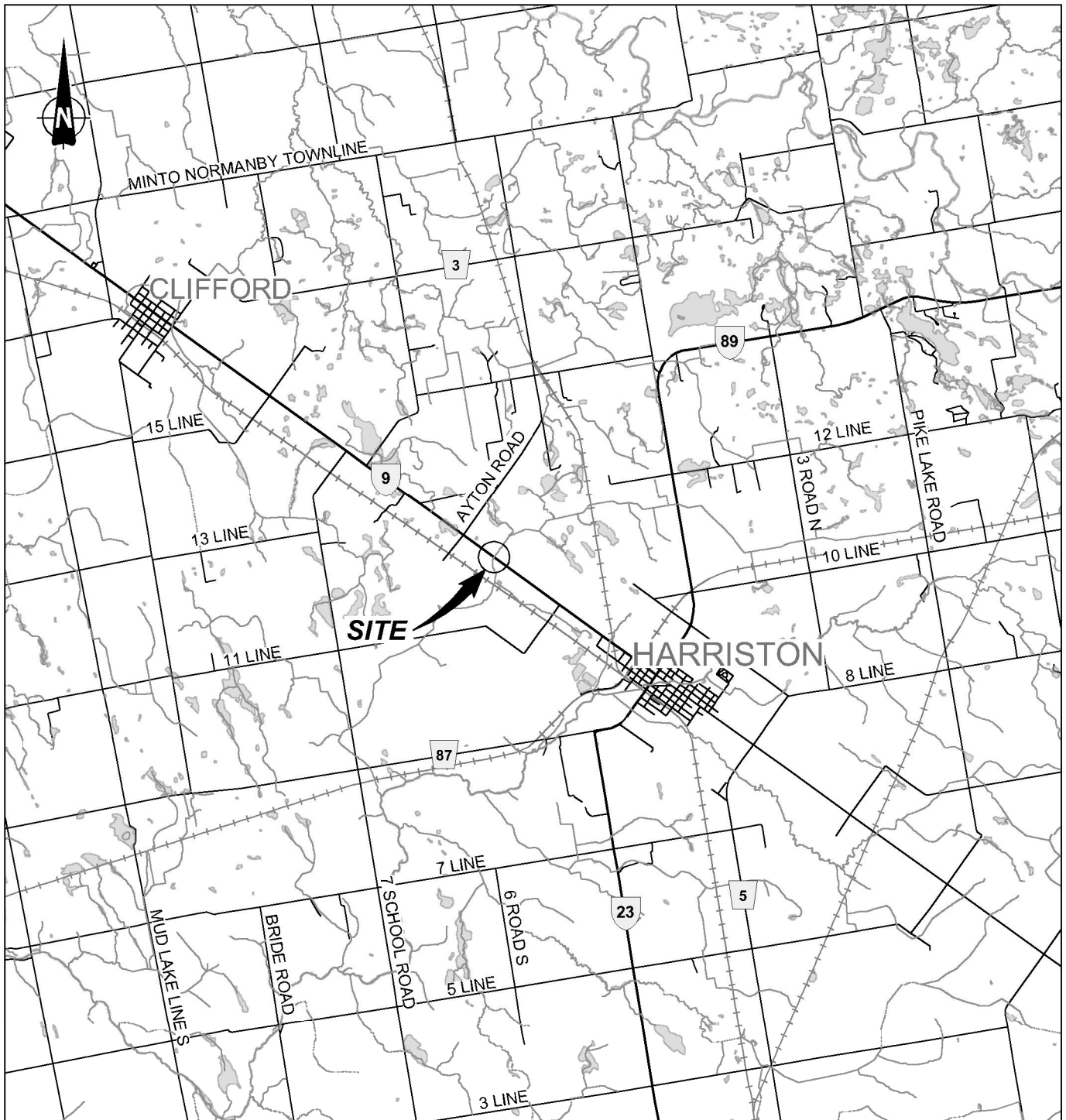
1 OF 1

METRIC

PROJECT 12-1132-0163
W.P. 3042-11-00 LOCATION N 4866686.1, E 191889.7 ORIGINATED BY DH
DIST HWY 9 BOREHOLE TYPE POWER AUGER, HOLLOW STEM COMPILED BY ZJB/LMK
DATUM GEODETIC DATE September 27, 2016 CHECKED BY

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa										WATER CONTENT (%)		
								○ UNCONFINED ● QUICK TRIAXIAL	+ FIELD VANE × LAB VANE											
376.48	GROUND SURFACE					▽														
0.00	FILL, sand and crushed gravel Brown																			
376.05																				
0.43	FILL, sandy silt, some clay, trace to some gravel, with topsoil layers, wood pieces and cobbles Loose to compact Brown		1	SS	15															
			2	SS	6															
			3	SS	16															
373.58																				
2.90	SILTY SAND AND GRAVEL Compact Brown		4	SS	20															
372.82																				
3.66	SANDY SILT, trace to some clay, trace gravel Compact Brown		5	SS	22															
372.06																				
4.42	SILTY SAND AND GRAVEL Dense to very dense Brown		6	SS	60															
			7	SS	38															
370.54																				
5.94	SANDY SILT TILL, trace to some clay, trace to some gravel, with cobbles Dense to very dense Brown to grey at about elev. 369.8m		8	SS	41															
			9	SS	91															
			10	SS	50/ 50mm															

[illegible]



REFERENCE

PLAN BASED ON CANMAP STREETFILES V.2008.5.

NOTE

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

PROJECT

DRAIN 1 CULVERT REPLACEMENT, SITE 35-594/C
HIGHWAY 9
GWP 3042-11-00

TITLE

KEY PLAN



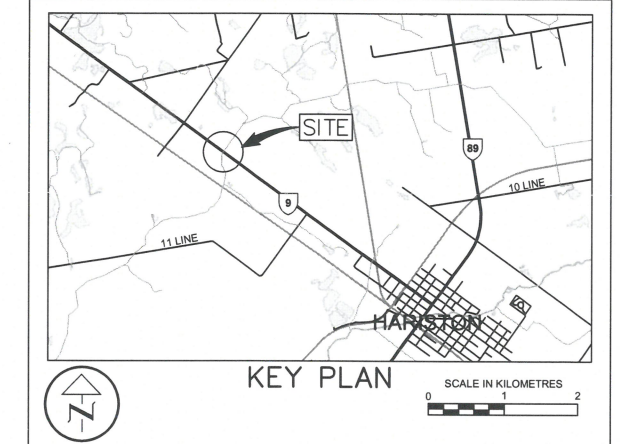
PROJECT No.		12-1132-0163	FILE No.	1211320163-4000-F06001	
CADD	ZJB/LMK	Jan. 12/17	SCALE	AS SHOWN	REV. 0
CHECK			FIGURE 1		

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

CONT No.
WP No. 3042-11-00



DRAIN 1 CULVERT REPLACEMENT
HIGHWAY 9
BOREHOLE LOCATIONS AND SOIL STRATA



- LEGEND**
- Borehole - Current Investigation
 - Seal
 - Piezometer
 - N Standard Penetration Test Value
 - 16 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
 - WL measured on January 5, 2017
 - WL encountered during drilling

No.	ELEVATION	CO-ORDINATES (MTM ZONE 10)	
		NORTHING	EASTING
BH-601	376.47	4 866 683.7	191 903.2
BH-602	376.48	4 866 686.1	191 889.7
BH-603	374.49	4 866 665.4	191 903.5

NOTES

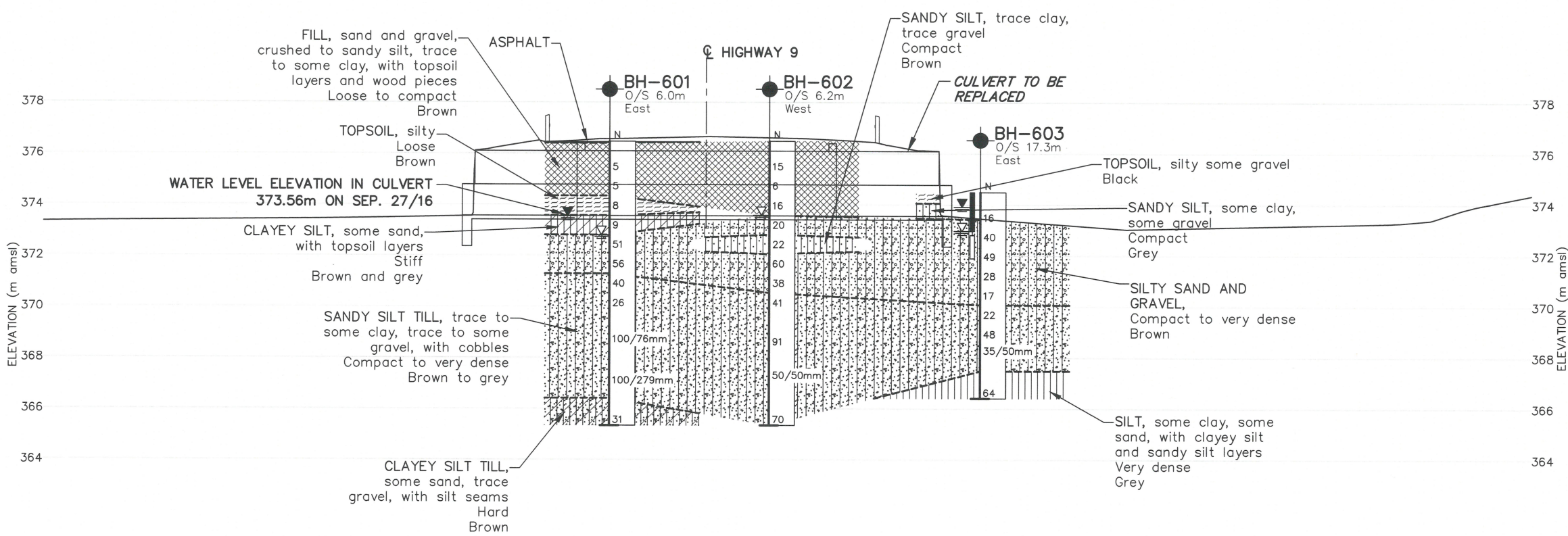
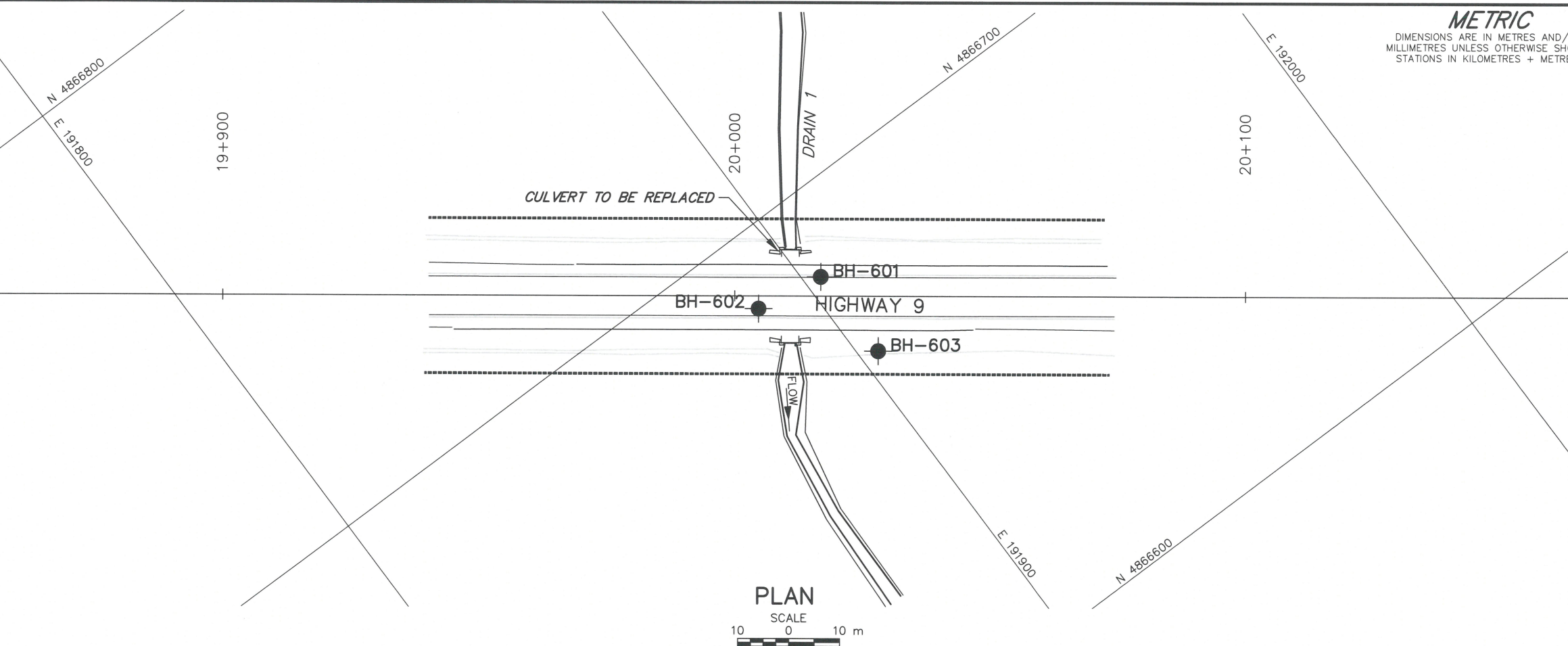
This drawing is for subsurface information only. Surface details and features are for conceptual illustration.

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

REFERENCE

Base plans provided in digital format by Stantec.

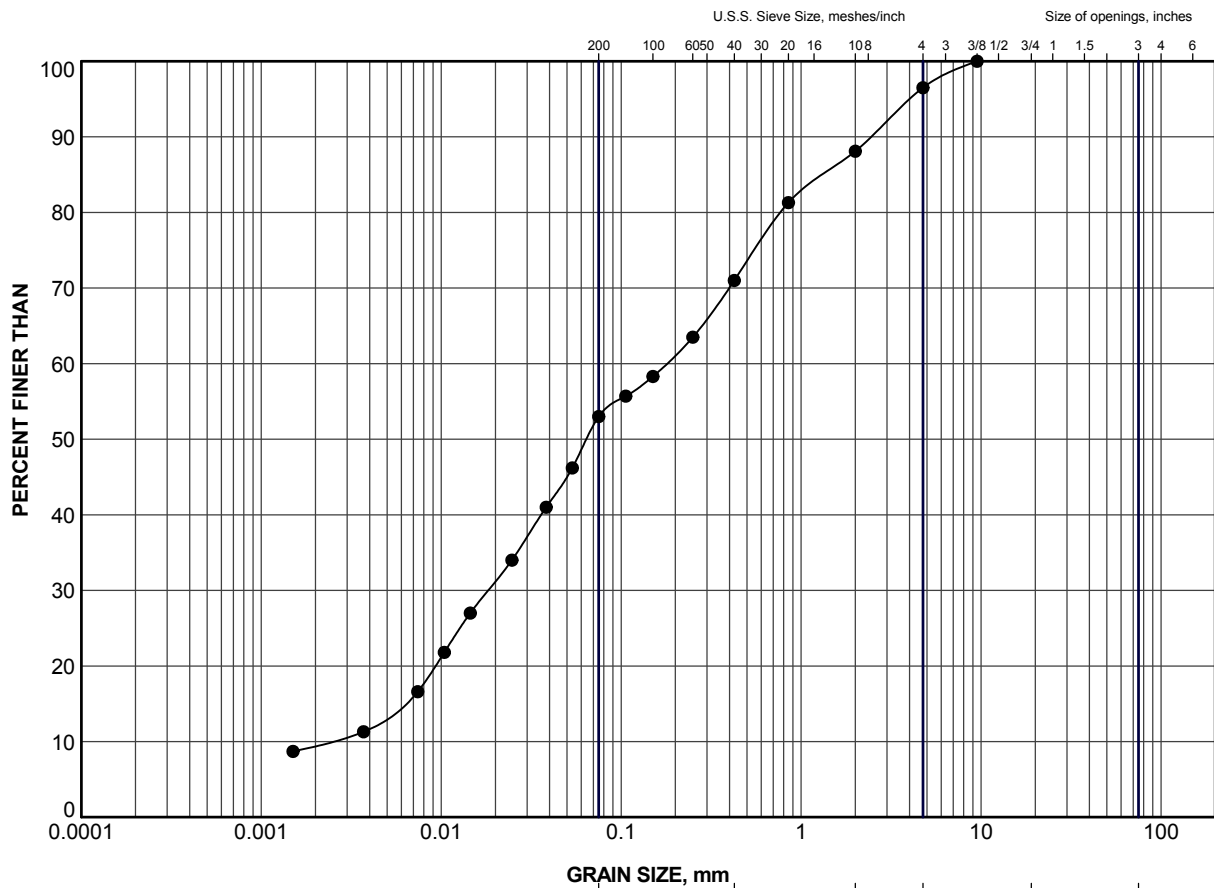
NO.	DATE	BY	REVISION
Geocres No. 40P15-48			
HWY.	9	PROJECT NO.	12-1132-0163
SUBM'D.	BT	CHKD.	DH
DRAWN:	LMK	CHKD.	DUP
DATE:	Mar 16/17	APPD.	FJH
SITE:	35-594/C	DWG.	1






APPENDIX A

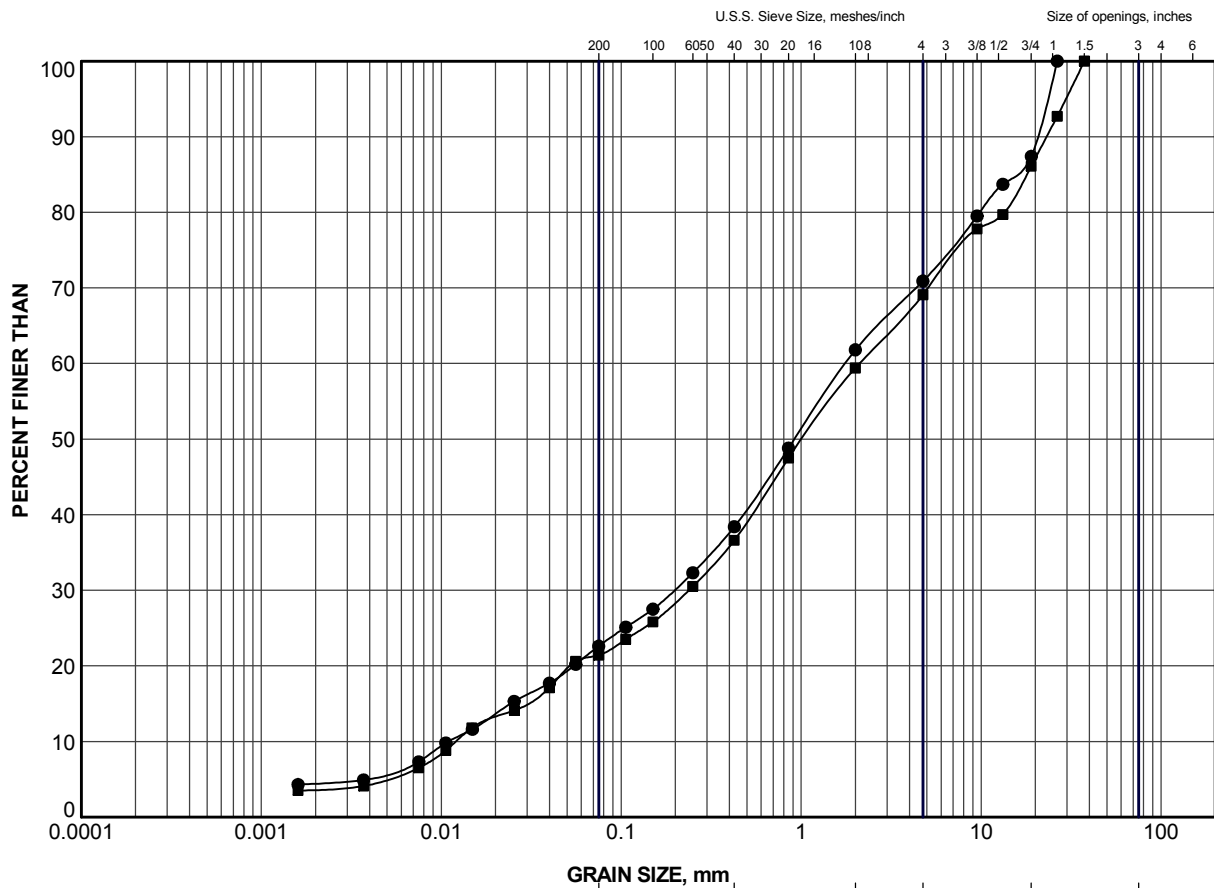
Laboratory Test Data



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	BH-601	1	375.5

PROJECT			
DRAIN 1 CULVERT REPLACEMENT, SITE NO. 35-594/C HIGHWAY 9 GWP 3042-11-00			
TITLE			
GRAIN SIZE DISTRIBUTION FILL			
PROJECT No.		12-1132-0163	FILE No. 1211320163-4000-F060A1
DRAWN		ZJB	Oct 21/16
CHECK			
		SCALE	N/A
		REV.	
		FIGURE A-1	

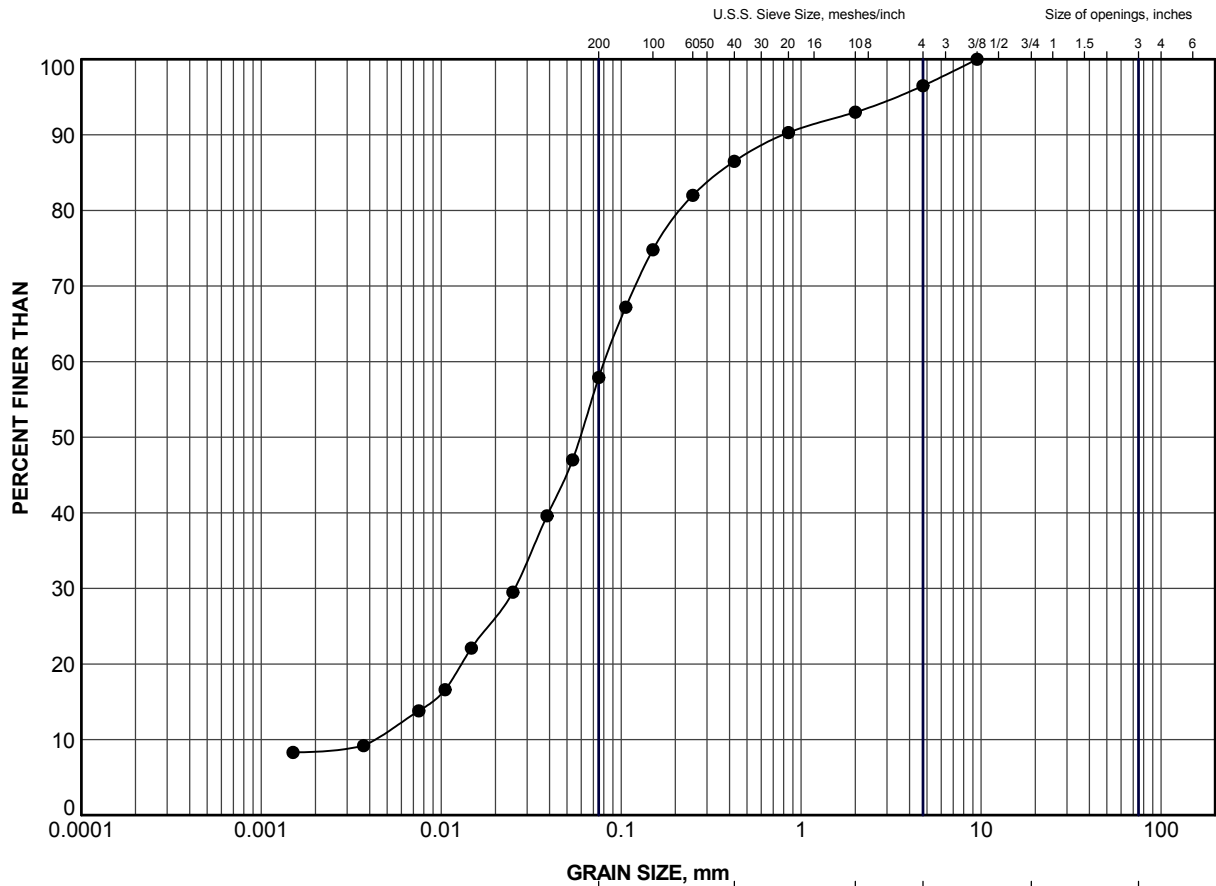


CLAY AND SILT	GRAVEL SIZE, mm					Cobble Size
	fine	medium	coarse	fine	coarse	
	SAND SIZE			GRAVEL SIZE		

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	BH-601	6	371.7
■	BH-603	3	372.0

PROJECT			
DRAIN 1 CULVERT REPLACEMENT, SITE NO. 35-594/C HIGHWAY 9 GWP 3042-11-00			
TITLE			
GRAIN SIZE DISTRIBUTION SILTY SAND AND GRAVEL			
PROJECT No. 12-1132-0163		FILE No. 1211320163-4000-F060A2	
SCALE N/A		REV.	
DRAWN	ZJB	Feb 13/17	
CHECK			
Golder Associates		FIGURE A-2	

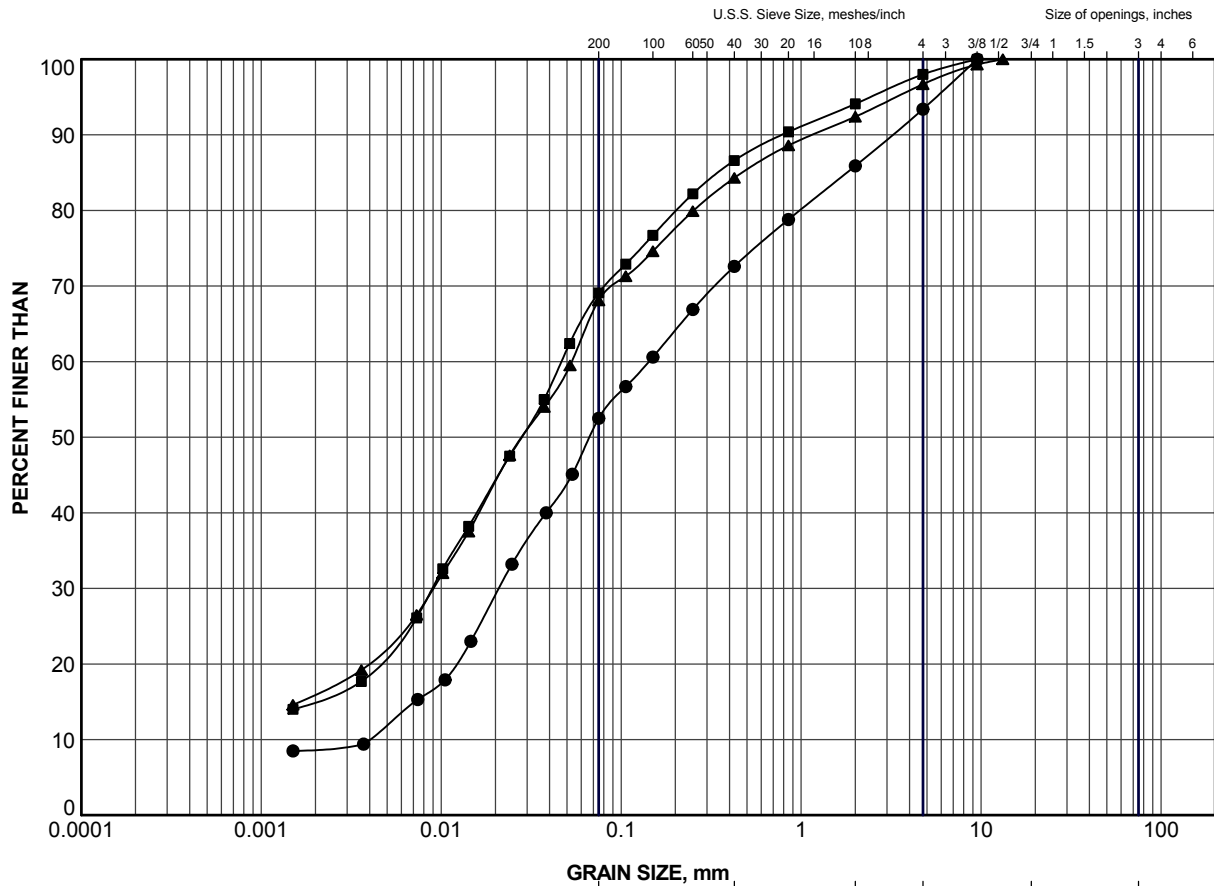


CLAY AND SILT	fine	medium	coarse	fine	coarse	Cobble Size
	SAND SIZE			GRAVEL SIZE		

LEGEND			
SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	BH-602	5	372.4

PROJECT			
DRAIN 1 CULVERT REPLACEMENT, SITE NO. 35-594/C			
HIGHWAY 9			
GWP 3042-11-00			
TITLE			
GRAIN SIZE DISTRIBUTION			
SANDY SILT			
	PROJECT No.	12-1132-0163	FILE No.1211320163-4000-F060A3
	DRAWN	ZJB	Oct 21/16
	CHECK		
	SCALE	N/A	REV.
			FIGURE A-3


LDN_MTO_GSD_GLDR_LDN.GDT 21/10/16

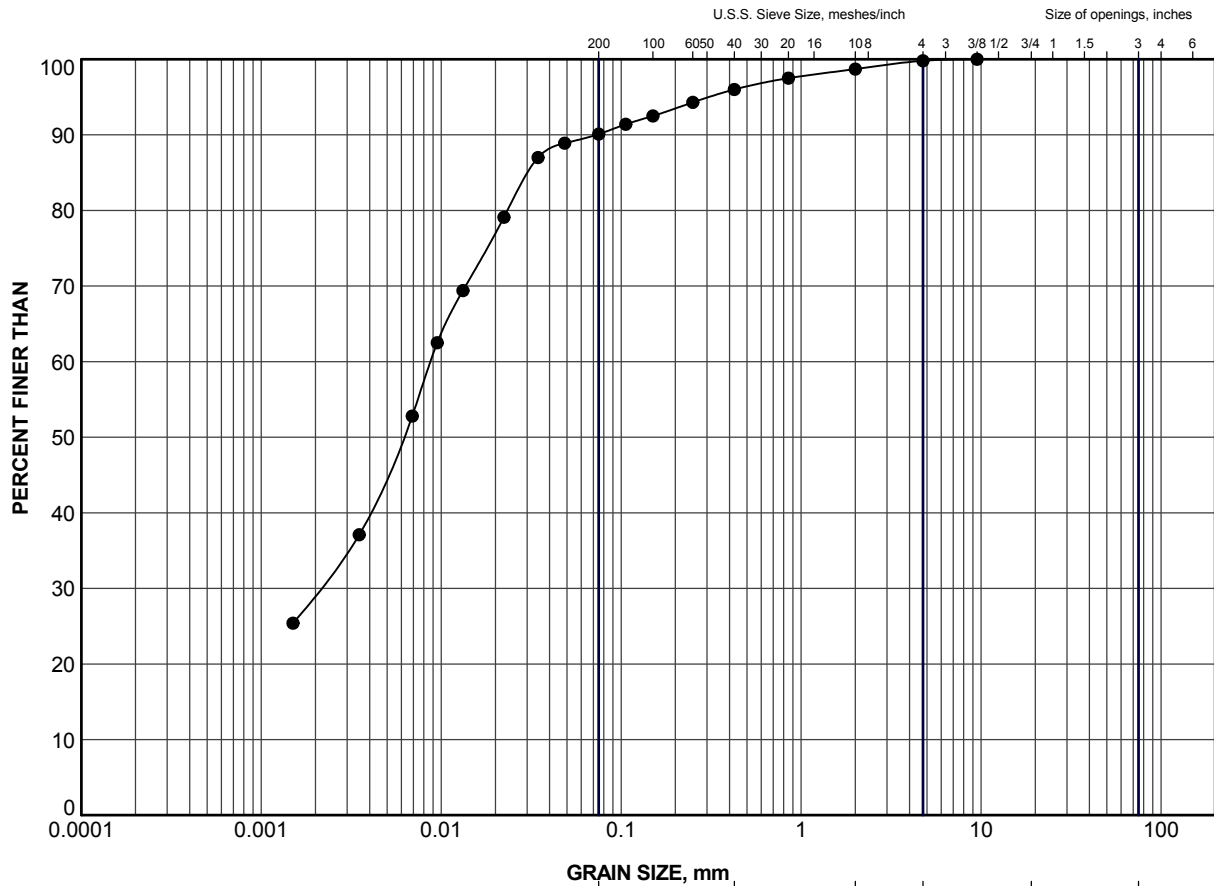


CLAY AND SILT		SAND SIZE, mm			GRAVEL SIZE, mm		Cobble Size
		fine	medium	coarse	fine	coarse	
		SAND SIZE			GRAVEL SIZE		

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	BH-601	8	370.2
■	BH-602	9	368.6
▲	BH-603	7	368.9


PROJECT			
DRAIN 1 CULVERT REPLACEMENT, SITE NO. 35-594/C HIGHWAY 9 GWP 3042-11-00			
TITLE			
GRAIN SIZE DISTRIBUTION SANDY SILT TILL			
PROJECT No.		12-1132-0163	FILE No.1211320163-4000-F060A4
DRAWN		ZJB	Oct 21/16
CHECK			
		SCALE	N/A
		REV.	
		FIGURE A-4	

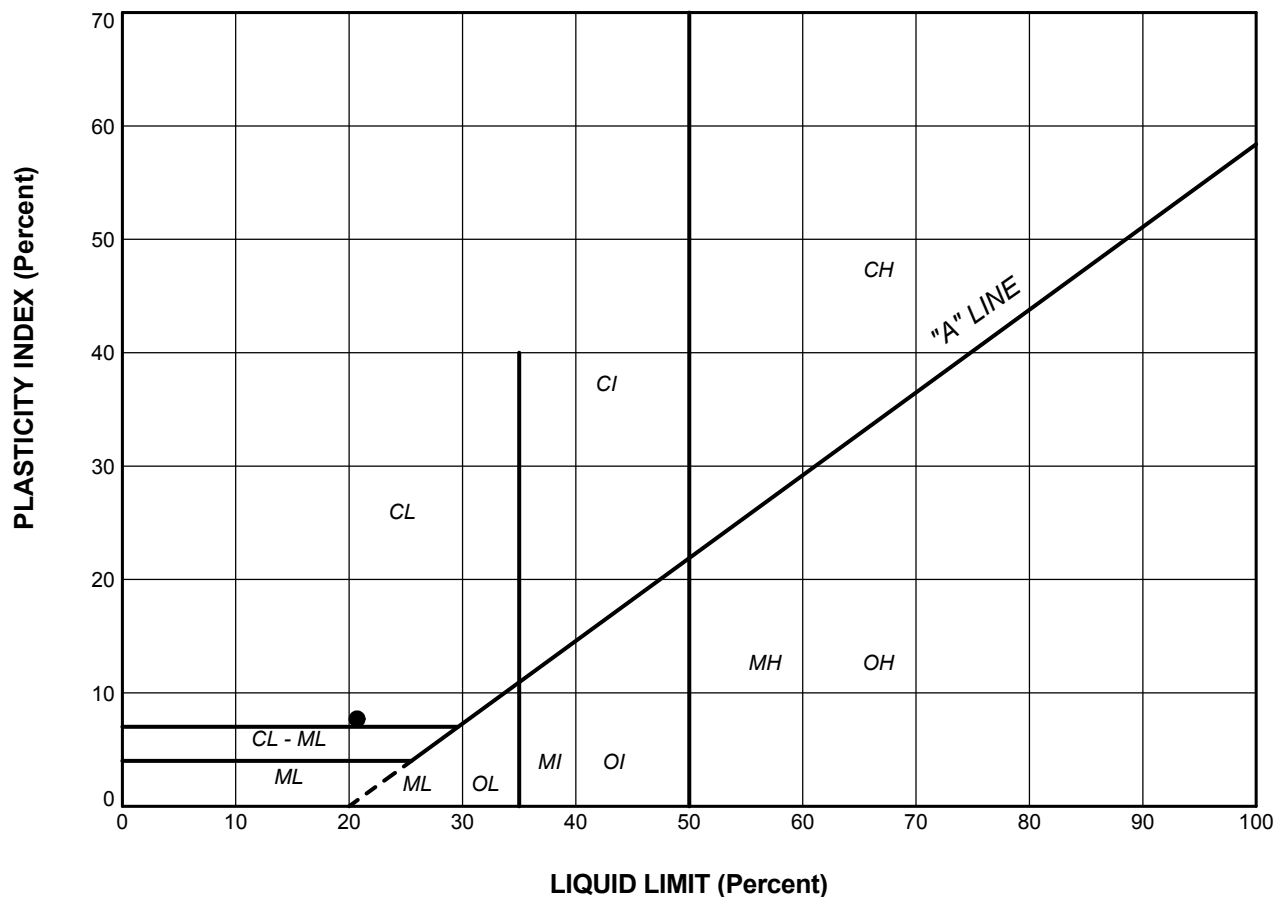


CLAY AND SILT	fine	medium	coarse	fine	coarse	Cobble Size
	SAND SIZE			GRAVEL SIZE		

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	BH-601	11	365.6

PROJECT				DRAIN 1 CULVERT REPLACEMENT, SITE NO. 35-594/C HIGHWAY 9 GWP 3042-11-00			
TITLE				GRAIN SIZE DISTRIBUTION CLAYEY SILT TILL			
PROJECT No.		12-1132-0163		FILE No.		1211320163-4000-F060A5	
DRAWN		ZJB		SCALE		N/A	
CHECK				REV.			
		Oct 21/16					
				FIGURE A-5			



LEGEND

SYMBOL	BOREHOLE	SAMPLE	LL(%)	PL(%)	PI
●	BH-601	11	20.7	13.0	7.7

CLAYEY SILT TILL

PROJECT
DRAIN 1 CULVERT REPLACEMENT, SITE NO. 35-594/C
HIGHWAY 9
GWP 3042-11-00

TITLE

PLASTICITY CHART



PROJECT No.	12-1132-0163	FILE No	1211320163-4000-F060A6
DRAWN	ZJB	Oct 21/16	SCALE N/A REV.
CHECK			

FIGURE A-6



APPENDIX B

Site Photographs



APPENDIX B PHOTOGRAPHS



Photograph 1: North elevation (inlet) of Culvert Site 35-594/C.



Photograph 2: South elevation (outlet) of Culvert Site 35-594/C.



APPENDIX B PHOTOGRAPHS



Photograph 3: Looking west along Highway 9 at Culvert Site 35-594/C.

n:\active\2012\1132 - geo\1132-0100\12-1132-0163 stantec-fdns-mega culverts-3011-e-0041\ph 4000-gwp 3042-11-00\rpts\r06 35-594c drain 1\1211320163-4000-r06 apr 3 17 (final) app b
- photos.docx



APPENDIX C

NSSPs



APPENDIX C

NSSP – COBBLES AND BOULDERS

NOTICE TO CONTRACTOR – Soil Conditions - Item No.

Non-Standard Special Provision

The Contractor is alerted to the presence of cobbles and boulders within the fill and native cohesive till soils. All associated work relating to this shall be included in the applicable tender items.

BASIS OF PAYMENT

Payment at the contract price for this Tender Item shall include full compensation for all labour, equipment and material required to do the work.

n:\active\2012\1132 - geo\1132-0100\12-1132-0163 stantec-fdns-mega culverts-3011-e-0041\ph 4000-gwp 3042-11-00\rpts\r06 35-594c drain 1\1211320163-4000-r06 apr 3 17 (final) app c
nssp cobbles and boulders.docx



APPENDIX C NSSP – WORKING SLAB

WORKING SLAB, Item No.

Non-Standard Special Provision

SCOPE

This Special Provision covers the requirements for the supply and placement of concrete working slab under the structure foundations. The purpose of the working slab is to protect the subgrade from disturbance and loosening due to construction traffic and ponded water and also to provide a level working surface.

CONSTRUCTION

Protection of Founding Soil

- Following inspection and approval of the prepared subgrade, a working slab with a minimum thickness of 100 millimetres shall be placed on the foundation subgrade should the subgrade remain exposed for a prolonged period, as per the contract drawings and documents. The concrete shall have a minimum 28 day compressive strength of 20 MPa.

BASIS OF PAYMENT

Payment at the contract price for this Tender Item shall include full compensation for all labour, equipment and material required to do the work.

n:\active\2012\1132 - geo\1132-0100\12-1132-0163 stantec-fdns-mega culverts-3011-e-0041\ph 4000-gwp 3042-11-00\rpts\r06 35-594c drain 1\1211320163-4000-r06 apr 3 17 (final) app c
nssp working slab.docx

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