

**FOUNDATION INVESTIGATION AND DESIGN REPORT
GRANDVIEW CREEK CULVERT – LANDFILL ROAD
HIGHWAY 11/17 - FOUR LANING
FROM 0.36 km EAST OF HIGHWAY 527 EASTERLY 12.6 km
TO 1 km WEST OF MACKENZIE STATION ROAD
G.W.P. 623-89-00, SITE 48C-351/C**

Geocres Number: 52A-159

Report to

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PART 1: FACTUAL INFORMATION

1 INTRODUCTION

This report presents the factual findings obtained from a foundation investigation conducted at the proposed location of the Grandview Creek culvert under the new Landfill Road in the Township of MacGregor, District of Thunder Bay. The new roadway and culvert are planned as part of the proposed Highway 11/17 four-laning project extending from 0.36 km east of Highway 527 to 1 km west of MacKenzie Station Road. The existing Highway 11/17 will become the new eastbound lanes of the four-lane divided highway, new westbound lanes will be constructed, and the new Landfill Road will be constructed parallel to and north of the highway.

The purpose of this investigation was to explore the subsurface conditions at the site and, based on the data obtained, to provide a borehole location plan, records of boreholes, stratigraphic sections, laboratory test results and written descriptions of the subsurface conditions. A model of the subsurface conditions was developed from the data obtained in the course of the investigation.

Thurber carried out the investigation as a sub-consultant to McCormick Rankin Corporation, under the Ministry of Transportation Ontario (MTO) Agreement Number 6009-E-0017.

2 SITE DESCRIPTION

The proposed Grandview Creek culvert is located approximately 14 km east of Thunder Bay, Ontario and approximately 10.5 km east of Highway 527. The new culvert will be situated approximately 100 m north of the existing Highway 11/17 alignment. The existing roadway embankment is approximately 5 to 6 m in height.

Grandview Creek flows from north to south at the proposed culvert location. Lands surrounding the culvert site consist of forested areas with bedrock outcrops. Cobbles and boulders were observed within the creek channel and at ground surface at various locations across the site.

Photographs in Appendix C show the general nature of the site.

The site lies near the border of the Superior and Southern Geological Provinces of the Canadian Shield. According to bedrock geology maps produced by the Ontario Geological Survey, the culvert site lies near a boundary between massive granodiorite to granite rocks and metasedimentary rocks. Bedrock core samples confirm that the site is underlain by fine grained metasedimentary rocks. Locally, the overburden consists of cohesionless deposits of silty to gravelly sand and silty sand till, as well as clayey silt till.

3 SITE INVESTIGATION AND FIELD TESTING

The site investigation and field testing for this project were carried out between November 8 and 16, 2011. Six boreholes, identified as GCL-1 to GCL-6, were drilled and sampled at the site.

Boreholes GCL-1 and GCL-4 were located near the proposed culvert inlet, Boreholes GCL-2 and GCL-5 were located near the centreline of the proposed roadway, and Boreholes GCL-3 and GCL-6 were located near the culvert outlet. The approximate borehole locations are shown on the attached Borehole Locations and Soil Strata drawing included in Appendix G.

The boreholes were advanced to depths of 1.0 m to 9.1 m (elevations 241.8 to 233.8). Bedrock was proven in Boreholes GCL-1, GCL-2, GCL-3 and GCL-6 by coring 3.0 m to 3.9 m into bedrock.

The borehole locations were marked in the field and utility clearances were obtained prior to drilling. Clearing and access preparation were required prior to commencement of the borehole drilling. Silt fencing was installed between the drill area and the creek to prevent migration of core water sediment into the adjacent creek.

A track mounted CME 45 drill rig was used at this site and a combination of hollow-stem augers, casing and NQ coring techniques were used to advance the boreholes. Overburden samples were obtained at selected intervals using a split spoon sampler in conjunction with Standard Penetration Testing (SPT). All rock cores were logged, and the Total Core Recovery (TCR), Rock Quality Designation (RQD) and the Fracture Indices (FI) were determined.

The drilling and sampling operations were supervised on a full time basis by a member of Thurber's technical staff. The supervisor logged the boreholes and processed the recovered soil samples and rock cores for transport to Thurber's laboratory for further examination and testing.

Groundwater conditions were observed in the open boreholes during and upon completion of the drilling operations. Groundwater conditions observed after completion of coring were not representative of site conditions as water was introduced into the borehole during coring. A standpipe piezometer was installed in Borehole GCL-2 for subsequent monitoring of groundwater levels. The completion details of the piezometer and boreholes are summarized in Table 3.1. The piezometer was decommissioned in general accordance with MOE Regulation 903 in late July 2012.

Table 3.1 – Piezometer and Borehole Completion Details

Borehole	Borehole Depth/ Elevation (m)	Completion Details
GCL-1	-	Backfilled with bentonite holeplug to 3.1 m, then holeplug and cuttings to surface.
GCL-2	5.2 / 237.4	Bentonite holeplug from 8.4 m to 5.2 m. Piezometer installed at 5.2 m. Filter sand from 5.2 m to 3.2 m, then bentonite holeplug to surface.
GCL-3	-	Backfilled with bentonite holeplug to 4.6 m, then holeplug and auger cuttings to surface.
GCL-4	-	Backfilled with auger cuttings to surface.
GCL-5	-	Backfilled with auger cuttings to surface.
GCL-6	-	Backfilled with bentonite holeplug to surface.

4 LABORATORY TESTING

The recovered soil samples were subjected to Visual Identification (VI) and to natural moisture content determination. Selected samples were also subjected to gradation analysis and Atterberg Limits testing where appropriate. The results of these tests are summarized on the Record of Borehole sheets included in Appendix A and are presented on the figures included in Appendix B.

Point load tests were carried out on selected samples of intact bedrock upon arrival at the laboratory to evaluate the unconfined compressive strength (UCS) of the bedrock. The UCS values of the rock assessed from the point load data are reported on the borehole logs.

5 DESCRIPTION OF SUBSURFACE CONDITIONS

Reference is made to the Record of Borehole sheets included in Appendix A. Details of the encountered soil stratigraphy are presented in these sheets and on the “Borehole Locations and Soil Strata” drawing included in Appendix G. An overall description of the stratigraphy is given in the following paragraphs. However, the factual data presented in the Record of Borehole sheets governs any interpretation of the site conditions.

The subsurface stratigraphy encountered at the site varied notably from the west to the east sides of the culvert alignment. Along the east side, bedrock was encountered at shallow depth below topsoil and silty sand layers. Along the west side, bedrock was encountered at greater depths and the stratigraphy consisted of a topsoil layer overlying variable deposits of silty to gravelly sand, silty sand till, clayey silt till, and locally clayey silt and peat layers. More detailed descriptions of the individual strata are presented below.

5.1 Topsoil

Topsoil was encountered at the surface in all boreholes. The topsoil is dark brown to black and described as silty to sandy. The topsoil thickness varied from 200 mm to 800 mm.

SPT N-values recorded in the topsoil ranged from 3 to 6 blows for 0.3 m penetration, indicating a very loose to loose relative density. Moisture contents of 33% to 97% were measured.

5.2 Clayey Silt

A 400 mm thick layer of clayey silt was encountered below the topsoil locally in Borehole GCL-1. The clayey silt was brown and contained some sand and trace gravel. The lower boundary of the layer was encountered at 1.2 m depth (elevation 241.9).

An SPT N-value of 7 blows for 0.3 m penetration was recorded in the clayey silt, indicating a firm consistency. The moisture content of a sample was 16%.

5.3 Peat

A 200 mm thick layer of peat was encountered below the clayey silt in Borehole GCL-1. The bottom of the peat was encountered at 1.4 m depth (elevation 241.7). A sample of the peat had a moisture content of 158%.

5.4 Gravelly Sand

A gravelly sand layer was encountered locally beneath the peat in Borehole GCL-1. The gravelly sand was brown and contained some silt and trace clay. This layer was 0.7 m thick with the bottom of the layer encountered at 2.1 m depth (elevation 241.0).

A single SPT was conducted in the gravelly sand layer which yielded an N-value of 92 blows for 0.3 m penetration, indicating a very dense relative density. A moisture content taken from a sample of the gravelly sand measured 19%.

One sample of the gravelly sand underwent laboratory grain size analysis testing, the results of which are summarized below. These results are also presented on the Record of Borehole sheets in Appendix A and the grain size distribution curve for this sample is plotted on Figure B1, Appendix B.

Gravel %	32
Sand %	50
Silt %	15
Clay %	3

5.5 Silty Sand

A layer of silty sand was encountered below the gravelly sand in Borehole GCL-1 and below the topsoil in Boreholes GCL-2, GCL-4, GCL-5 and GCL-6. The silty sand was brown to grey and contained trace to some gravel and trace clay.

The thickness of the silty sand layer ranged from 0.3 m to 0.9 m, with the lower boundary of the layer encountered at depths of 0.6 to 3.0 m (elevations 242.7 to 240.1).

SPT N-values recorded in the silty sand ranged from 15 to 44 blows for 0.3 m penetration, indicating a compact to dense relative density. An SPT N-value of 62 blows for 0.2 m penetration was recorded in Borehole GCL-5 and is attributed to SPT refusal on bedrock.

The moisture content of the silty sand ranged from 14% to 27%.

Two samples of the silty sand were selected for grain size distribution testing. The results of these tests are summarized on the Record of Borehole sheets in Appendix A and the grain size distribution curves for these samples are plotted on Figure B2, Appendix B. The results are as follows:

Gravel %	0 to 11
Sand %	51 to 58
Silt %	34 to 35
Clay %	3 to 8

5.6 Silty Sand to Silt Till

Till was encountered beneath the silty sand layer in Borehole GCL-2 and below the topsoil in Borehole GCL-3. In Borehole GCL-2, the till was grey and consisted of silt, some clay to clayey, with some sand and trace gravel. In Borehole GCL-3, the till was brown and consisted of silty sand containing some gravel and trace clay. Occasional cobbles and boulders were encountered in the till.

The silt till in Borehole GCL-2 was 3.8 m thick, with a lower boundary at 5.2 m depth (elevation 237.4). The silty sand till in Borehole GCL-3 was 5.5 m thick, with a base at 6.1 m depth (elevation 236.8).

SPT N-values recorded in the till typically ranged from 57 blows for 0.3 m penetration to 50 blows for 0.05 m penetration, indicating a very dense relative density. Lower N-values of 12 and 20 blows per 0.3 m were recorded above 2.0 m depth in Borehole GCL-3, indicating a compact condition. Coring was required to advance Borehole GCL-2 through a boulder encountered in the till.

The moisture content of samples of the till ranged from 10% to 20%.

One sample of the silt till and one sample of the silty sand till underwent laboratory gradation analysis, the results of which are summarized below. These results are also presented on the Record of Borehole sheets included in Appendix A and are plotted on Figures B3 and B4, Appendix B. Atterberg Limits testing was conducted on the silt till sample, and the results are plotted on Figure B5 in Appendix B.

	<u>Silt Till</u>	<u>Silty Sand Till</u>
Gravel %	1	15
Sand %	17	54
Silt %	68	28
Clay %	14	3
Liquid Limit	22	-
Plastic Limit	15	-

5.7 Bedrock

Bedrock and auger refusal on probable bedrock were encountered below the silty sand and till in all boreholes. The bedrock was proven by coring in Boreholes GCL-1 to GCL-3 and GCL-6. Additional auger probes were advanced to refusal adjacent to Boreholes GCL-4 and GCL-5 to confirm depth of probable bedrock. The depths and elevations of bedrock encountered in the boreholes are summarized in Table 5.1.

Table 5.1 – Depth to Bedrock/Probable Bedrock at Borehole Locations

Borehole	Depth to Bedrock (m)	Top of Bedrock Elevation (m)	Method of Confirming
GCL-1	3.0	240.1	Cored
GCL-2	5.2	237.4	Cored
GCL-3	6.1	236.8	Cored
GCL-4	1.1	241.8	Auger refusal
GCL-5	1.0	241.8	Auger refusal
GCL-6	0.6	242.7	Cored

The bedrock recovered in the cores was described as greenish grey metasedimentary bedrock with occasional white quartz veins.

Total core recovery in Boreholes GCL-1, GCL-2 and GCL-6 ranged from 83% to 100% with the exception of the initial runs commenced above the bedrock surface (50 to 55% recovery). Total core recovery in Borehole GCL-3 was 63 and 59%.

RQD values ranged from 0% to 72%, indicating very poor to fair rock quality. The Fracture Index (FI) of the rock, expressed as fractures per 0.3 m of core, was generally less than 10.

The unconfined compressive strength of the rock, estimated from the results of point load tests conducted on the rock core samples, typically ranged from 59 to 197 MPa, indicating a strong to very strong intact rock. The results are summarized on the Record of Borehole sheets in Appendix A (as average per run).

5.8 Water Levels

Water levels were observed in the open boreholes prior to commencement of coring operations. Water was added to the boreholes during coring operations and therefore water levels were not recorded in these boreholes upon completion.

A standpipe piezometer was installed in Borehole GCL-2 following completion of drilling. The ground water depths and levels measured in the piezometer and in the open boreholes prior to coring are summarized in Table 5.2.

Table 5.2 – Water Level Measurements

Borehole	Date	Water Level (m)		Comment
		Depth	Elevation	
GCL-1	Nov. 09, 2011	1.2	241.9	During drilling
GCL-2	Jan. 30, 2012	1.1	241.5	Piezometer
GCL-4	Nov. 16, 2011	0.5	242.4	During drilling
GCL-5	Nov. 16, 2011	0.9	241.9	During drilling

The water depth in the creek at the time of the fieldwork was variable but generally in the order of 0.2 m. Based on this observation and the creek invert levels shown on the preliminary design drawings, the creek water level is estimated to be near elevation 242.2.

The above values are short-term readings and seasonal fluctuations of the groundwater level are to be expected. In particular, the groundwater level may be at a higher elevation after the spring snowmelt or after periods of heavy rainfall, and will reflect the water level in the creek.

6 MISCELLANEOUS

The borehole locations were selected by Thurber Engineering Ltd. and staked in the field by McCormick Rankin Corporation (MRC). The co-ordinates and ground surface elevations at the boreholes were surveyed by MRC. Where boreholes required relocation from the staked location, field measurements were recorded and the surveyed coordinates and elevations adjusted accordingly.

Thurber obtained utility clearances for the borehole locations prior to drilling.

Eastern Ontario Diamond Drilling Ltd. from Hawkesbury, Ontario supplied a track mounted CME 45 drill rig and conducted the drilling, sampling and in-situ testing operations.

The field program was supervised on a full time basis by Mr. Ryan Kromer, E.I.T. of Thurber.

Routine laboratory testing was carried out by Thurber Engineering Ltd.

Overall supervision of the field program was conducted by Mr. Mark Farrant, P.Eng. Interpretation of the data and preparation of this report were carried out by Ms. Lindsey Blaine, E.I.T. and Ms. Mei Cheong, M.Phil.

The report was reviewed by Mr. Murray R. Anderson, M.Eng., P.Eng. and Dr. P.K. Chatterji, P.Eng. a Designated Principal Contact for MTO Foundations Projects.

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PART 2: ENGINEERING DISCUSSION AND RECOMMENDATIONS

7 INTRODUCTION

This report presents interpretation of the geotechnical data in the factual report and presents geotechnical recommendations for design of the new culvert carrying Grandview Creek under the new Landfill Road. The culvert installation is part of the Highway 11/17 four-laning project, in which the existing highway will become the new eastbound lanes of the four-lane divided highway, new westbound lanes will be constructed, and Landfill Road will be constructed north of the highway.

The proposed culvert consists of a concrete arch culvert with a span of 12.8 m, a rise of 4.0 m and a length of 27.4 m (preliminary General Arrangement drawing dated June 2012). The culvert will be supported on precast concrete footings with a thickness of 0.6 m and a design top of footing level at Elev. 243.4 (north/inlet) to Elev. 243.3 (south/outlet). The proposed culvert design was selected on the basis of considerations other than foundations.

The embankment height at the proposed culvert location will be in the order of 6 to 7 m with a proposed finished road grade at Elev. 250.0.

The discussions and recommendations presented in this report are based on the factual data obtained during the course of the investigation. The preliminary General Arrangement drawings used for preparation of this report were provided by Hatch Mott MacDonald.

8 CULVERT FOUNDATIONS

The subsurface stratigraphy encountered at the site varies notably from the west to the east sides of the culvert alignment. Along the east side, bedrock and probable bedrock was encountered at depths of 0.6 m to 1.1 m (elevation 241.8 to 242.7) below topsoil and silty sand layers. Along the

east side, bedrock was encountered at depths of 3.0 to 6.1 m (elevation 236.8 to 240.1) and the stratigraphy consisted of a topsoil layer overlying variable deposits of silty to gravelly sand, silty sand till, clayey silt till, and locally clayey silt and peat layers.

The groundwater level at the site is expected to be near the water level in the creek, estimated at approximate elevation 242.2 at the time of drilling.

Foundation recommendations for design of poured in-place or precast spread footings to support the proposed culvert are provided in the following sections. Comments regarding alternative foundation systems (steel piles, augered caissons) are also presented in the event that the design concept changes.

A comparison of the foundation alternatives based on advantages and disadvantages of each is included in Appendix D. A foundation scheme preferred from a foundations perspective is recommended.

8.1 Spread Footings on Native Soils

The anticipated founding level for spread footings supporting an open footing culvert, assuming 2.2 m of frost cover between the footing base and the finished ground surface within the culvert (top of footing from preliminary GA drawing), will range from Elev. 241.2 at the inlet to Elev. 241.1 at the outlet. Along the east side of the culvert however, bedrock was encountered above these levels and therefore the founding levels will be higher. Based on the borehole information, the soil/bedrock conditions at the founding level will consist of the following:

Table 8.1 – Anticipated Conditions at Founding Level

Location		Borehole	Founding Level	Anticipated Foundation Subgrade
West Side	Inlet	GCL-1	241.2	Dense gravelly/silty sand
	Middle	GCL-2	241.2	Very dense silt till
	Outlet	GCL-3	241.1	Very dense silty sand till
East Side	Inlet	GCL-4	241.8	Bedrock
	Middle	GCL-5	241.8	Bedrock
	Outlet	GCL-6	242.7	Bedrock

The following geotechnical resistances are recommended for design of spread footings founded on the bedrock or dense to very dense native soils at the above founding levels:

	Sand/Till			Bedrock
Footing Width (m)	<u>0.9</u>	<u>1.2</u>	<u>1.5</u>	
Factored Geotechnical Resistance at ULS (kPa)	450	525	600	2,000
Geotechnical Resistance at SLS (kPa)	450	425	400	N/A

The width of footing must be designed based on the load demand from the culvert structure and overlying embankment fill.

The geotechnical resistances are based on a footing subjected to vertical concentric loading. Where eccentric or inclined loads are applied, the resistance used in the design must be reduced in accordance with the CHBDC Clause 6.7.3 and 6.7.4.

The geotechnical resistance at SLS provided for footings on native soil is based on an estimated total settlement not exceeding 25 mm. The SLS values take into consideration the potential for some disturbance of the founding surface during excavation for footing construction “in the wet”. The geotechnical resistance at SLS will not govern design of footings founded on bedrock, and negligible settlement is expected.

The anticipated founding levels are up to 1.1 m below the approximate creek and groundwater levels. In view of the proximity of the footings to the creek, the high permeability of the soils, and the presence of cobbles and boulders potentially obstructing installation of sheet pile shoring, dewatering of the excavation and construction of the west culvert footings in the dry at the design founding level is likely to be impractical. Therefore, construction of spread footings will require subexcavation to the design level below water (“in the wet”) in short sections of about 2 m length followed by immediate placement of concrete using tremie methods.

Consideration could be given to raising the founding levels and providing frost protection for the founding surfaces using equivalent thermal insulation. However, the use of insulation is generally not practical for culvert foundations. Further, excavation to depths of about 1.1 m below the creek/groundwater levels would still be required to extend the footings to competent native soils. From this viewpoint, use of higher founding levels is not the recommended option.

The lateral resistance of the footings may be computed using an unfactored friction coefficient of 0.55 on the native sand/till. For footings on bedrock, an unfactored friction coefficient of 0.7 is recommended. These values require a degree of sliding movement to occur to fully mobilize the resistance.

The structural designers must ensure that the geometry of the proposed footing and the limits of subexcavation do not encroach into the creek.

8.2 Spread Footings on Rock Fill

In view of the high groundwater conditions, the high permeability of the soils, and the impracticality of dewatering excavations for footing construction, placement of spread footings on compacted rock fill may be considered as an option to establish the top of footing level above the water level.

Where constructed on soil, the rock fill must be a minimum 0.5 m thick and be placed on native, compact to very dense sand/till. Accordingly, the base of the rock fill must be placed no higher than the elevations indicated in Table 8.2, and deeper as required to provide a minimum 0.5 m thickness of rock fill below the base of the footing. The minimum thickness requirement does not apply to rock fill placed directly on bedrock.

Table 8.2 – Highest Level for Underside of Rock Fill

Location		Borehole	Highest Recommended Base Level	Underlying Soil
West Side	Inlet	GCL-1	241.7	Very dense gravelly sand
	Middle	GCL-2	242.0	Dense silty sand
	Outlet	GCL-3	241.4	Compact silty sand till
East Side	Inlet	GCL-4	241.8	Bedrock
	Middle	GCL-5	241.8	Bedrock
	Outlet	GCL-6	242.7	Bedrock

Rock fill placement will generally be carried out below the water level, and should involve subexcavation in short sections followed by immediate backfilling to above the water level to permit placement of the footings in the dry. The rock fill should be placed in accordance with OPSS 206 including compaction by several passes of heavy tracked equipment.

The recommended gradation of the rock fill is as follows:

<u>Sieve Size</u>	<u>Percent Passing</u>
150 mm	100
106 mm	50 – 100
75 mm	15 – 80
26.5 mm	0 – 15

A minimum 150 mm thick layer of compacted 19 mm clear stone should be placed above rock fill to provide an even founding surface for placement of the footings. Details of footing construction on rock fill are presented in Figure F1, Appendix F.

The geotechnical resistances recommended for design of spread footings founded on a minimum 0.5 m thickness of rock fill are as follows:

Footing Width (m)	<u>0.9</u>	<u>1.2</u>	<u>1.5</u>
Factored Geotechnical Resistance at ULS (kPa)	450	525	600
Geotechnical Resistance at SLS (kPa)	450	425	400

The geotechnical resistances are based on a footing subjected to vertical concentric loading. The width of footing must be designed based on the load demand from the culvert structure and overlying embankment fill. Where eccentric or inclined loads are applied, the resistance used in the design must be reduced in accordance with the CHBDC Clause 6.7.3 and 6.7.4.

The geotechnical resistance at SLS provided is based on an estimated total settlement on the culvert structure not exceeding 25 mm.

The lateral resistance of the footings may be computed using an unfactored friction coefficient of 0.6 assuming a friction angle of 31° between the footing concrete and underlying clear stone. This value requires a degree of sliding movement to occur to fully mobilize the resistance.

8.3 Driven Steel Piles

The native soils at this site are typically dense to very dense and contain cobbles and boulders. Use of rock coring equipment was required to penetrate cobbles and boulders locally. In addition, bedrock was encountered at relatively shallow depths below the culvert level. The use of driven steel H-piles or sheet piles is not recommended in these conditions and these alternatives have not been further developed.

8.4 Augered Caissons

Installation of caissons at this site is not recommended due to the presence of cobbles and boulders as well as the potential for base and sidewall instability in the cohesionless soils below the groundwater level. In view of these factors, this alternative has not been further developed.

8.5 Recommended Foundation

From a geotechnical perspective and based on the subsurface conditions, spread footings on rock fill placed to raise the founding level above the groundwater level are considered the most cost effective and practical foundation option for supporting the culvert type selected for this site. This option will enable footing construction above the water level, reduce excavation and dewatering requirements, and provide a more uniform founding surface than footings on native soils.

Alternative culvert types (ie., box culvert) may be preferable at this site based solely on foundation design and construction considerations. However, selection of the proposed culvert type was based on considerations other than foundations.

8.6 Frost Cover

The depth of frost penetration at this site is 2.2 m. The base of all footings on native soil must be provided with a minimum of 2.2 m of earth cover as protection against frost action. Frost protection is not required for footings on bedrock or for footings constructed on a minimum 0.5 m layer of rock fill placed to establish founding levels above the groundwater level.

9 CULVERT BACKFILL AND LATERAL EARTH PRESSURES

Culvert backfill should consist of free-draining granular material conforming to OPSS Granular A, Granular B Type II or Granular B Type III specifications.

Backfill should be placed and compacted in simultaneous equal lifts on both sides of the culvert, and the top of backfill elevation should be within 400 mm on both sides of the culvert at all times. Heavy compaction equipment should not be used adjacent to the walls and roof of the culvert. Compaction should be carried out in accordance with OPSS 501.

In general, earth pressures acting on the culvert walls may be assumed to impose a triangular distribution governed by the characteristics of the backfill. For a fully drained condition, the pressures should be computed in accordance with the CHBDC but generally are given by the expression:

$$p = K (\gamma h + q)$$

where: p = horizontal pressure on the wall at depth h (kPa)
 K = earth pressure coefficient (see Table 9.1)
 γ = bulk unit weight of retained soil (see Table 9.1)
 h = depth below top of fill where pressure is computed (m)
 q = value of any surcharge (kPa)

Earth pressure coefficients for backfill to the culvert are dependent on the material used as backfill. Recommended unfactored values are shown in Table 9.1. The at-rest coefficients should be employed for restrained culvert walls. Active pressures should be used for any wingwalls or unrestrained walls.

The parameters in the table correspond to full mobilization of active and passive earth pressures, and require certain relative movements between the wall and adjacent soil to produce these conditions. The values to be used in design can be assessed from Figure C6.16 of the Commentary to the CHBDC.

Table 9.1 – Earth Pressure Coefficients (K)

Condition	Earth Pressure Coefficient (K)			
	OPSS Granular A or Granular B Type II $\phi = 35^\circ, \gamma = 22.8 \text{ kN/m}^3$		OPSS Granular B Type I or Type III $\phi = 32^\circ, \gamma = 21.2 \text{ kN/m}^3$	
	Horizontal Surface Behind Wall	Sloping Backfill (2H:1V)	Horizontal Surface Behind Wall	Sloping Backfill (2H:1V)
Active (Unrestrained Wall)	0.27	0.38*	0.31	0.46*
At Rest (Restrained Wall)	0.43	-	0.47	-
Passive	3.7	-	3.3	-

* For wing walls.

In accordance with Clause 6.9.3 of the CHBDC, a compaction surcharge should be added. The magnitude should be 12 kPa at the top of fill and decreasing to 0 kPa at a depth of 2.0 m for Granular B Type I or at a depth of 1.7 m for Granular A or Granular B Type II.

The design of the culvert must incorporate measures such as weepholes or subdrains to permit drainage of the culvert backfill, or alternatively the culvert walls should be designed to withstand the potential build-up of hydrostatic pressures behind the walls.

10 EROSION CONTROL

Erosion and scour protection must be provided for the culvert foundations. In general, this will involve placing the footings below the level of potential scour and/or providing rock protection over the footings to prevent erosion and undermining of the foundations. Design of the erosion protection measures must consider hydrologic and hydraulic concerns and should be carried out by specialists experienced in this field.

A vegetation cover should be established on all other exposed earth surfaces to protect against surficial erosion, in general accordance with OPSS 804.

11 EXCAVATION AND GROUNDWATER CONTROL

The excavation and backfilling for foundations must be carried out in accordance with OPSS 902. Construction staging will include temporary detouring of traffic onto the new WBL during culvert construction and therefore roadway protection will not be required.

All excavation must be carried out in accordance with the Occupational Health and Safety Act (OHSA). For the purposes of the OHSA, the native sand/till above the water table may be classed as Type 3 soil. This classification is based on the lack of cohesion in the soils. The cohesionless soils below the water table are classified as Type 4 soil.

Excavation for footing construction and/or placement of rock fill to prepare the founding surface is expected to extend up to about 1.1 m below the groundwater levels within cohesionless soils containing cobbles and boulders. In these conditions, installation of sheet pile shoring, dewatering of the excavation and construction of culvert footings in the dry within close proximity to the creek may be impractical.

The recommended procedure for preparation of the founding surface entails subexcavation in the wet to the depths outlined in Section 8 in short sections of about 2 m length followed by immediate backfilling with rock fill to the required founding level (allowing for the clear stone layer) and/or placement of tremie concrete. The contractor must ensure that the excavation does not encroach into the creek by controlling the length of excavation open at any one time.

Selection of the equipment and methodology to excavate and prepare the founding surface is the responsibility of the Contractor. The Contract Documents should contain a NSSP advising the Contractor of the high groundwater levels, cohesionless soils and cobbles and boulders at this site that may impact foundation construction. Suggested wording is provided in Appendix E.

12 SEISMIC CONSIDERATIONS

The following seismic parameters should be used for design:

- Velocity Related Seismic Zone 0
- Zonal Velocity Ratio 0.0
- Acceleration Related Seismic Zone 0
- Zonal Acceleration Ratio 0.0
- Peak Horizontal Acceleration 0.02

The soil profile type at this site has been classified as Type I. Therefore, according to Table 4.4 of the CHBDC, a Site Coefficient “S” (ground motion amplification factor) of 1.0 should be used in seismic design.

In accordance with Clause 4.6.4 of the CHBDC, retaining structures should be designed using earth pressure coefficients that incorporate the effects of earthquake loading. The seismic component of the earth pressure distribution is additional to the static earth pressure distribution and may be taken as an inverted triangle with the maximum pressure at the top of the wall and the minimum pressure at the toe. The seismic earth pressure parameters (ΔK_{AE}) recommended for determining the seismic component are presented in Table 12.1:

Table 12.1 – Earth Pressure Coefficients for Earthquake Loading

Condition	Seismic Earth Pressure Coefficient (ΔK_E)			
	OPSS Granular A or OPSS Granular B Type II $\phi = 35^\circ, \gamma = 22.8 \text{ kN/m}^3$		OPSS Granular B Type I or Type III $\phi = 32^\circ, \gamma = 21.2 \text{ kN/m}^3$	
	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)
Active (ΔK_{AE})*	0.01	0.03	0.01	0.03
At Rest (ΔK_{OE})**	0.02	-	0.03	-
Passive (ΔK_{PE})	-	-	-0.1	-

* After Mononobe and Okabe, passive case assumes a horizontal surface in front of the wall.

** After Woods

The foundation soils at the site are assessed as not being prone to liquefaction.

13 CONSTRUCTION CONCERNS

Potential construction concerns include, but are not necessarily limited to:

- Preparation of the founding surfaces for spread footings will require excavation below the groundwater level within cohesionless soils containing cobbles and boulders. This work will require excavation in short sections (in the wet) followed by immediate backfilling with rock fill or tremie concrete. Driving of sheet piling is not considered feasible.
- Large boulders may be encountered within the excavation depth. Removal of these boulders will require suitable excavating equipment, and may result in areas of over-excavation requiring additional rock fill to backfill.

The successful performance of the culvert will depend largely upon good workmanship and quality control during construction. Observation of the excavation and backfilling operations by the QVE will be required during construction to confirm that the foundation recommendations are correctly implemented and material specifications are met.


14 CLOSURE

Engineering analysis and preparation of the report were carried out by Ms. Mei T. Cheong, M.Phil.

The report was reviewed by Mr. Murray R. Anderson, P.Eng. and Dr. P.K. Chatterji, P.Eng. a Designated Principal Contact for MTO Foundations Projects.

Thurber Engineering Ltd

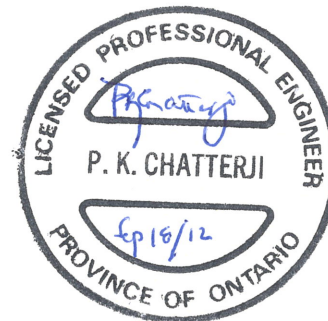
Mei T. Cheong, M.Phil.
Geotechnical Specialist


Sept 18/12

Murray R. Anderson, P.Eng., M.Eng.
Senior Foundations Engineer



P. K. Chatterji, P.Eng., Ph.D.
Review Principal



Appendix A

Record of Borehole Sheets

SYMBOLS, ABBREVIATIONS AND TERMS USED ON RECORDS OF BOREHOLES

1. TEXTURAL CLASSIFICATION OF SOILS

CLASSIFICATION	PARTICLE SIZE	VISUAL IDENTIFICATION
Boulders	Greater than 200mm	same
Cobbles	75 to 200mm	same
Gravel	4.75 to 75mm	5 to 75mm
Sand	0.075 to 4.75mm	Not visible particles to 5mm
Silt	0.002 to 0.075mm	Non-plastic particles, not visible to the naked eye
Clay	Less than 0.002mm	Plastic particles, not visible to the naked eye

2. COARSE GRAIN SOIL DESCRIPTION (50% greater than 0.075mm)

TERMINOLOGY	PROPORTION
Trace or Occasional	Less than 10%
Some	10 to 20%
Adjective (e.g. silty or sandy)	20 to 35%
And (e.g. sand and gravel)	35 to 50%

3. TERMS DESCRIBING CONSISTENCY (COHESIVE SOILS ONLY)

DESCRIPTIVE TERM	UNDRAINED SHEAR STRENGTH (kPa)	APPROXIMATE SPT ⁽¹⁾ 'N' VALUE
Very Soft	12 or less	Less than 2
Soft	12 to 25	2 to 4
Firm	25 to 50	4 to 8
Stiff	50 to 100	8 to 15
Very Stiff	100 to 200	15 to 30
Hard	Greater than 200	Greater than 30

NOTE: Hierarchy of Soil Strength Prediction

- 1) Laboratory Triaxial Testing
- 2) Field Insitu Vane Testing
- 3) Laboratory Vane Testing
- 4) SPT value
- 5) Pocket Penetrometer



4. TERMS DESCRIBING DENSITY (COHESIONLESS SOILS ONLY)

DESCRIPTIVE TERM	SPT "N" VALUE
Very Loose	Less than 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very Dense	Greater than 50

5. LEGEND FOR RECORDS OF BOREHOLES

SYMBOLS AND ABBREVIATIONS FOR SAMPLE TYPE	SS Split Spoon Sample	WS Wash Sample	AS Auger (Grab) Sample
	TW Thin Wall Shelby Tube Sample	TP Thin Wall Piston Sample	
	PH Sampler Advanced by Hydraulic Pressure	PM Sampler Advanced by Manual Pressure	
	WH Sampler Advanced by Self Static Weight	RC Rock Core	SC Soil Core

$$\text{Sensitivity} = \frac{\text{Undisturbed Shear Strength}}{\text{Remoulded Shear Strength}}$$


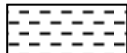



 Water Level
 Shear Strength Determination by Pocket Penetrometer

- (1) SPT 'N' Value Standard Penetration Test 'N' Value – refers to the number of blows from a 63.5kg hammer free falling a height of 0.76m to advance a standard 50 mm outside diameter split spoon sampler for 0.3 m depth into undisturbed ground.
- (2) DCPT Dynamic Cone Penetration Test – Continuous penetration of a 50 mm outside diameter, 60° conical steel point attached to "A" size rods driven by a 63.5 kg hammer free falling a height of 0.76 m. The resistance to cone penetration is the number of hammer blows required for each 0.3 m advance of the conical point into undisturbed ground.

UNIFIED SOILS CLASSIFICATION

MAJOR DIVISIONS		GROUP SYMBOL	TYPICAL DESCRIPTION
COARSE GRAINED SOILS	GRAVEL AND GRAVELLY SOILS	GW	Well-graded gravels or gravel-sand mixtures, little or no fines.
		GP	Poorly-graded gravels or gravel-sand mixtures, little or no fines.
		GM	Silty gravels, gravel-sand-silt mixtures.
		GC	Clayey gravels, gravel-sand-clay mixtures.
	SAND AND SANDY SOILS	SW	Well-graded sands or gravelly sands, little or no fines.
		SP	Poorly-graded sands or gravelly sands, little or no fines.
		SM	Silty sands, sand-silt mixtures.
		SC	Clayey sands, sand-clay mixtures.
FINE GRAINED SOILS	SILTS AND CLAYS $W_L < 50\%$	ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity.
		CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays. ($W_L < 30\%$).
		CI	Inorganic clays of medium plasticity, silty clays. ($30\% < W_L < 50\%$).
		OL	Organic silts and organic silty-clays of low plasticity.
	SILTS AND CLAYS $W_L > 50\%$	MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts.
		CH	Inorganic clays of high plasticity, fat clays.
		OH	Organic clays of medium to high plasticity, organic silts.
HIGHLY ORGANIC SOILS		Pt	Peat and other highly organic soils.
CLAY SHALE			
SANDSTONE			
SILTSTONE			
CLAYSTONE			
COAL			

EXPLANATION OF ROCK LOGGING TERMS

<u>ROCK WEATHERING CLASSIFICATION</u>		<u>SYMBOLS</u>	
Fresh (FR)	No visible signs of weathering.		
Fresh Jointed (FJ)	Weathering limited to the surface of major discontinuities.		CLAYSTONE
Slightly Weathered (SW)	Penetrative weathering developed on open discontinuity surfaces, but only slight weathering of rock material.		SILTSTONE
Moderately Weathered (MW)	Weathering extends throughout the rock mass, but the rock material is not friable.		SANDSTONE
Highly Weathered (HW)	Weathering extends throughout the rock mass and the rock is partly friable.		COAL
Completely Weathered (CW)	Rock is wholly decomposed and in a friable condition, but the rock texture and structure are preserved.		Bedrock (general)

<u>DISCONTINUITY SPACING</u>		<u>STRENGTH CLASSIFICATION</u>			
Bedding	Bedding Plane Spacing	Rock Strength	Approximate Uniaxial Compressive Strength		Field Estimation of Hardness*
			(MPa)	(psi)	
Very thickly bedded	Greater than 2m	Extremely Strong	Greater than 250	Greater than 36,000	Specimen can only be chipped with a geological hammer
Thickly bedded	0.6 to 2m				
Medium bedded	0.2 to 0.6m	Very Strong	100-250	15,000 to 36,000	Requires many blows of geological hammer to break
Thinly bedded	60mm to 0.2m	Strong	50-100	7,500 to 15,000	Requires more than one blow of geological hammer to break
Very thinly bedded	20 to 60mm				
Laminated	6 to 20mm	Medium Strong	25.0 to 50.0	3,500 to 7,500	Breaks under single blow of geological hammer.
Thinly Laminated	Less than 6mm				






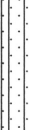

<u>TERMS</u>					
Total Core Recovery: (TCR)	Core recovered as a percentage of total core run length.	Weak	5.0 to 25.0	750 to 3,500	Can be peeled by a pocket knife with difficulty
Solid Core Recovery: (SCR)	Percent Ratio of solid core of full cylindrical shape recovered. Expressed with respect to the total length of core run.	Very Weak	1.0 to 5.0	150 to 750	Can be peeled by a pocket knife, crumbles under firm blows of geological pick.
Rock Quality Designation: (RQD)	Total length of sound core recovered in pieces 0.1m in length or larger as a percentage of total core run length.	Extremely Weak (Rock)	0.25 to 1.0	35 to 150	Indented by thumbnail
Uniaxial Compressive Strength (UCS)	Axial stress required to break the specimen				
Fracture Index: (FI)	Frequency of natural fractures per 0.3m of core run.				

RECORD OF BOREHOLE No GCL-1

1 OF 1

METRIC

W.P. 623-89-00 LOCATION N 5 376 355.4 E 378 274.0 Grandview Creek Landfill Rd. ORIGINATED BY RK
HWY 11/17 BOREHOLE TYPE Hollow Stem Augers/ NQ Coring COMPILED BY AN
DATUM Geodetic DATE 2011.11.09 - 2011.11.09 CHECKED BY RPR

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						
								20	40	60	80	100		
								○ UNCONFINED + FIELD VANE						
								● QUICK TRIAXIAL x LAB VANE						
								20	40	60	80	100		
									PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT			
									W _P	W	W _L			
									WATER CONTENT (%)					
									20	40	60			
243.1														
0.0	TOPSOIL , sandy Very Loose to Loose Dark Brown Moist		1	SS	4		243							
242.3														
0.8	Clayey SILT , some sand, trace gravel Firm Brown		2	SS	7									
241.9								242						
241.7	PEAT												156% >>>	
1.4	Gravelly SAND , some silt, trace clay Very Dense Brown Wet		3	SS	92									
241.0								241						
2.1	Silty SAND , trace gravel Dense Grey Wet		4	SS	38									
240.1														
3.0	METASEDIMENTARY BEDROCK , slightly weathered to fresh, greenish grey		1	RUN				240						
			2	RUN			239							
	Occasional quartz veins		3	RUN			238							
237.0														
6.1	END OF BOREHOLE AT 6.1m. WATER LEVEL OBSERVED AT 1.2m DURING DRILLING. BOREHOLE BACKFILLED WITH HOLEPLUG TO 3.1m, THEN HOLEPLUG AND AUGER CUTTINGS TO SURFACE.													

+³ X³: Numbers refer to
Sensitivity

20
15
10

(%) STRAIN AT FAILURE

METRIC

[illegible]

ONTMT4S 1182.GPJ 6/26/12

+³, ×³: Numbers refer to Sensitivity

METRIC

[illegible]

ONTMT4S 1182.GPJ 6/26/12

+³, ×³: Numbers refer to Sensitivity

RECORD OF BOREHOLE No GCL-4

1 OF 1

METRIC

W.P. 623-89-00 LOCATION N 5 376 353.2 E 378 283.1 Grandview Creek Landfill Rd. ORIGINATED BY RK
 HWY 11/17 BOREHOLE TYPE Hollow Stem Augers COMPILED BY AN
 DATUM Geodetic DATE 2011.11.16 - 2011.11.16 CHECKED BY RPR

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT	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 ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE								
242.9								20	40	60	80	100				
0.0	TOPSOIL, silty, some clay and sand Loose Dark Brown Moist to Wet		1	SS	4											
242.1																
0.8	Silty SAND, some gravel Compact Brownish Grey		2	SS	15		242									
241.8																
1.1	Wet															
END OF BOREHOLE AT 1.1m UPON AUGER REFUSAL. PROBING AT FIVE ADDITIONAL LOCATIONS WITHIN 2m OF THE BOREHOLE, ENCOUNTERED REFUSAL ON PROBABLE BEDROCK AT DEPTHS RANGING FROM 0.7m TO 1.1m. WATER OBSERVED AT 0.5m DURING DRILLING. BOREHOLE BACKFILLED WITH AUGER CUTTINGS TO SURFACE.																

ONTMT4S 1182.GPJ 6/25/12

RECORD OF BOREHOLE No GCL-5

1 OF 1

METRIC

W.P. 623-89-00 LOCATION N 5 376 347.7 E 378 281.9 Grandview Creek Landfill Rd. ORIGINATED BY RK
 HWY 11/17 BOREHOLE TYPE Hollow Stem Augers COMPILED BY AN
 DATUM Geodetic DATE 2011.11.16 - 2011.11.16 CHECKED BY RPR

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa								WATER CONTENT (%)		
242.8								20	40	60	80	100						
0.0	TOPSOIL, silty, occasional rootlets																	
242.5	Very Loose		1	SS	3													
0.3	Dark Brown																	
	Moist																	
241.8	Silty SAND, trace clay, occasional organics		2	SS	62/													0 58 34 8
1.0	Compact Brown				0.200													
	Moist to Wet																	
	END OF BOREHOLE AT 1.0m UPON AUGER REFUSAL. PROBING AT 1.2m SOUTH OF BOREHOLE, ENCOUNTERED PROBABLE BEDROCK AT 1.2m. WATER OBSERVED AT 0.9m DURING DRILLING. BOREHOLE BACKFILLED WITH AUGER CUTTINGS TO SURFACE.																	

+³, X³: Numbers refer to
Sensitivity

20
15
10
(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No GCL-6

1 OF 1

METRIC

W.P. 623-89-00 LOCATION N 5 376 333.4 E 378 282.5 Grandview Creek Landfill Rd. ORIGINATED BY RK
HWY 11/17 BOREHOLE TYPE Hollow Stem Augers/NQ Coring COMPILED BY AN
DATUM Geodetic DATE 2011.11.16 - 2011.11.16 CHECKED BY RPR

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 ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE				WATER CONTENT (%)						
243.3								20	40	60	80	100		20	40	60		
0.0	TOPSOIL, sandy, occasional rootlets Dark Brown Moist						243											
0.2																		
242.7	Silty SAND, trace clay Brown Moist		1	RUN														RUN #1 TCR=50% SCR=25% RQD=7% UCS=178MPa (Average)
0.6	METASEDIMENTARY BEDROCK, highly weathered, greenish grey, occasional quartz veins						242											5
			2	RUN														4
							241											3
			3	RUN														6
	Moderately weathered						240											0
			4	RUN														4
																		8
																		3
238.8							239											3
4.5	END OF BOREHOLE AT 4.5m. BOREHOLE BACKFILLED WITH HOLEPLUG TO SURFACE.																	

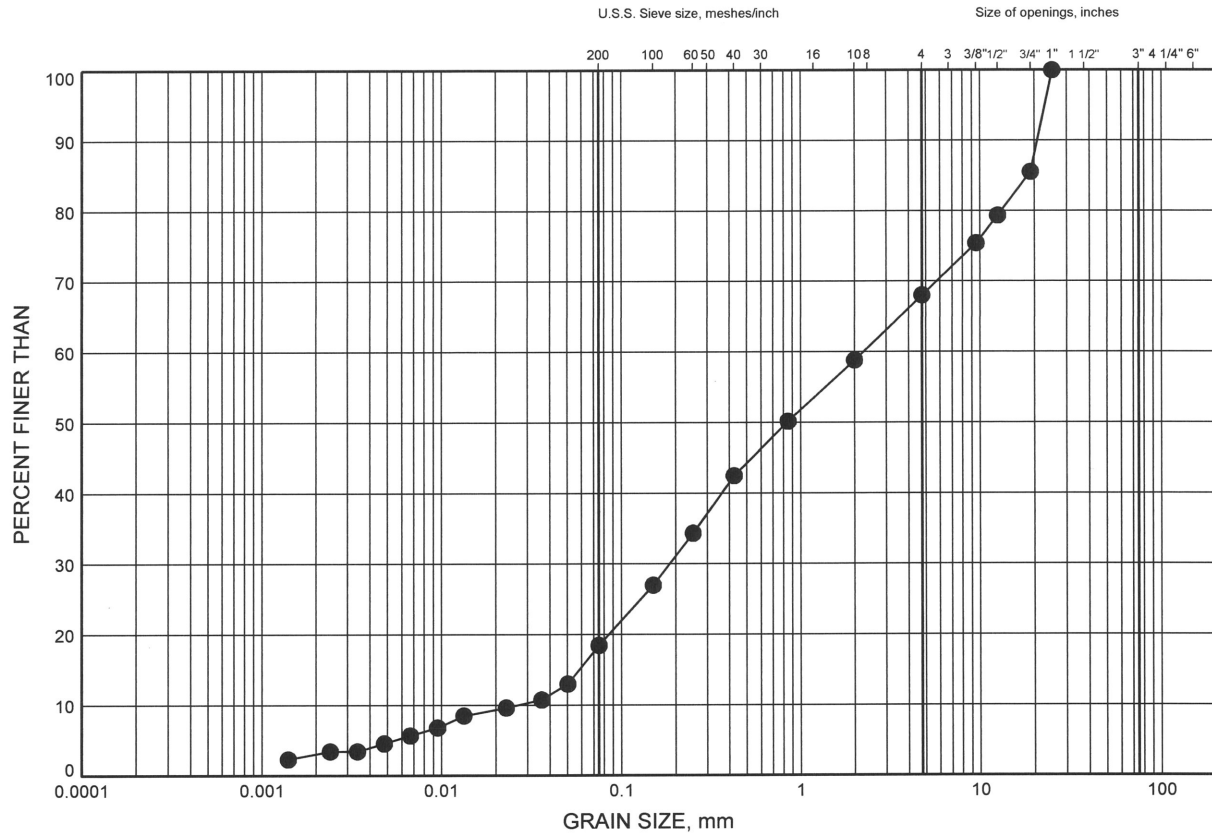
Appendix B

Laboratory Test Results

Grandview Creek Culvert at Landfill Rd.
GRAIN SIZE DISTRIBUTION

FIGURE B1

GRAVELLY SAND



SILT and CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
FINE GRAINED	SAND			GRAVEL		

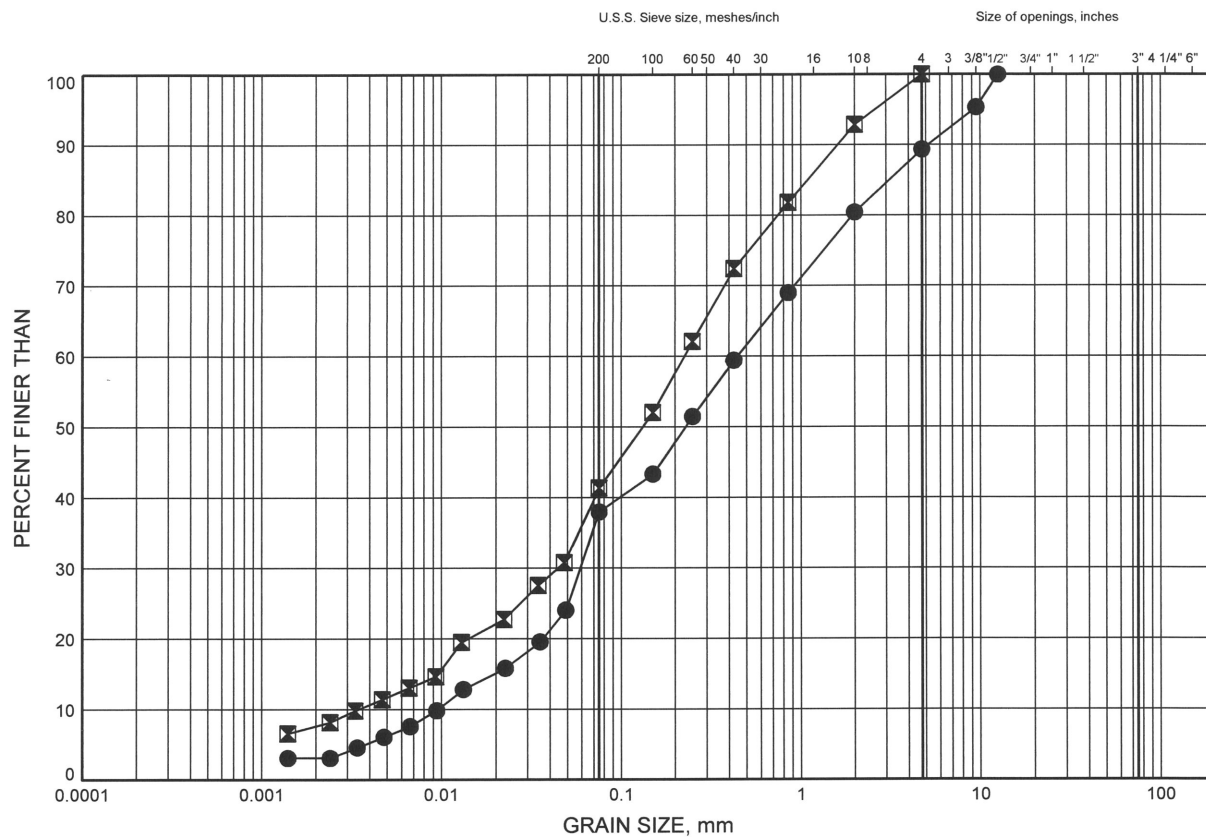
LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	GCL-1	1.83	241.27

Grandview Creek Culvert at Landfill Rd. GRAIN SIZE DISTRIBUTION

FIGURE B2

SILTY SAND



SILT and CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
FINE GRAINED	SAND			GRAVEL		

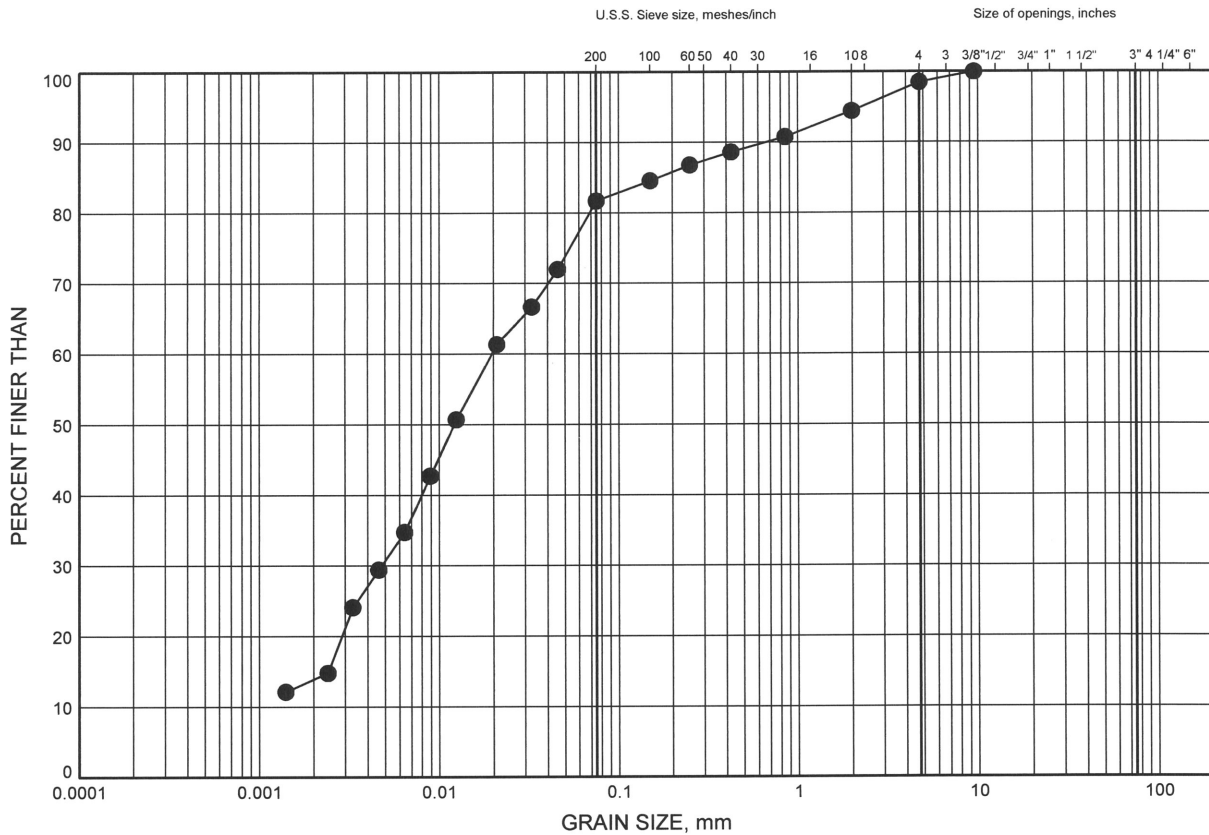
LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	GCL-2	0.91	241.69
◻	GCL-5	0.79	242.01

Grandview Creek Culvert at Landfill Rd. GRAIN SIZE DISTRIBUTION

FIGURE B3

SILT TILL



SILT and CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
FINE GRAINED	SAND			GRAVEL		

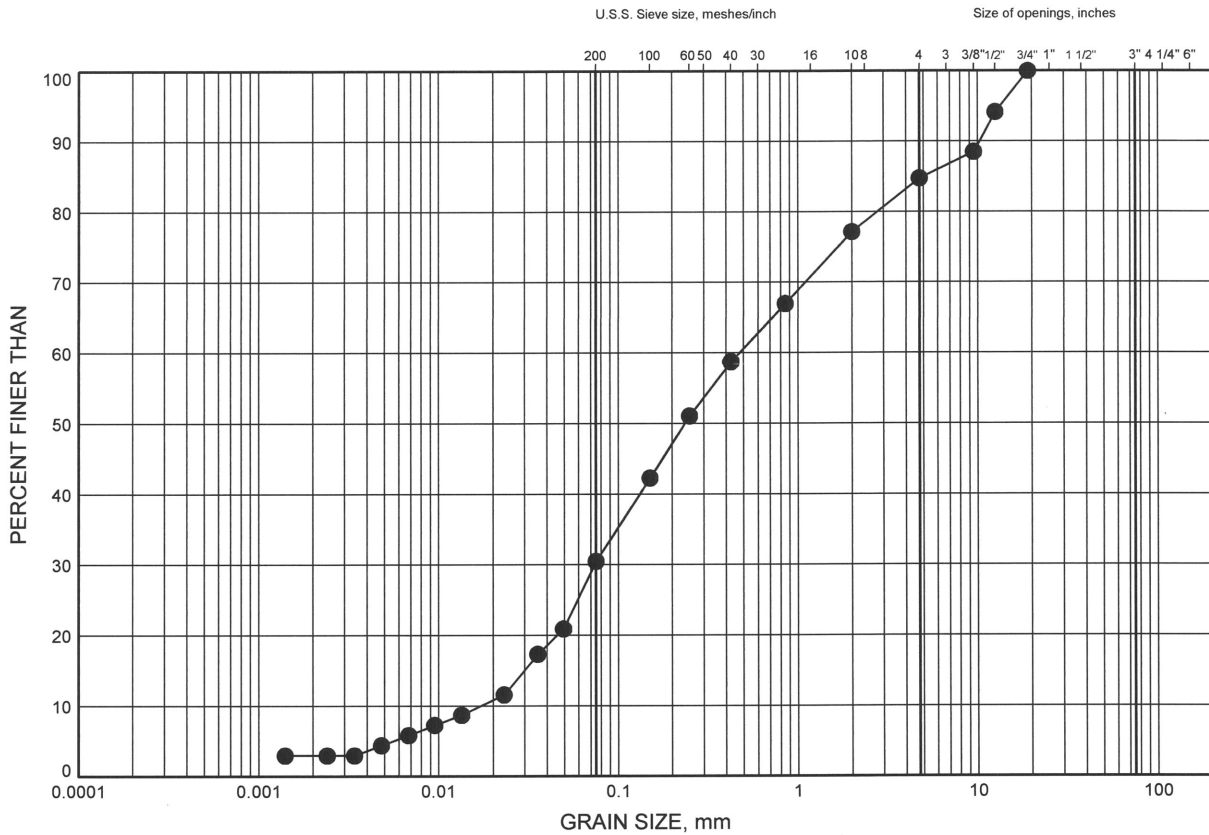
LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	GCL-2	1.83	240.77

Grandview Creek Culvert at Landfill Rd. GRAIN SIZE DISTRIBUTION

FIGURE B4

SILTY SAND TILL



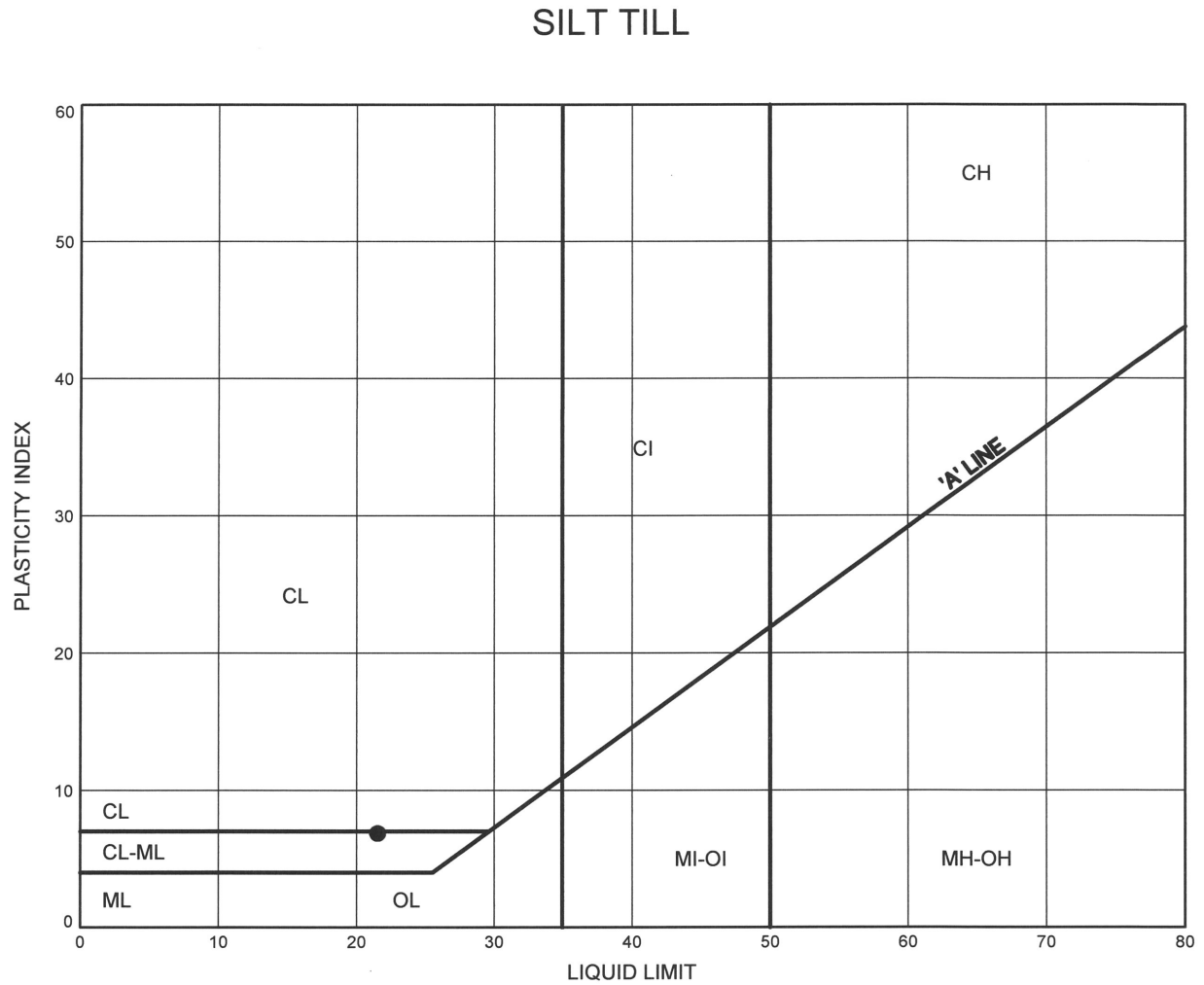
SILT and CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
FINE GRAINED	SAND			GRAVEL		

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	GCL-3	1.83	241.07

Grandview Creek Culvert at Landfill Rd.
ATTERBERG LIMITS TEST RESULTS

FIGURE B5



SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	GCL-2	1.83	240.77

Date January 2012
 Project 623-89-00



Prep'd AN
 Chkd. MC

Appendix C

Site Photographs



Photograph 1 – Grandview Creek looking east along proposed Landfill Road



Photograph 2 – Grandview Creek looking north towards Borehole GCL-1

Appendix D

Foundation Comparison

COMPARISON OF FOUNDATION ALTERNATIVES

Footings on Native Soil	Footings on Rock Fill	Driven Steel Piles	Caissons (Drilled Shaft)
<p>Advantages:</p> <ul style="list-style-type: none"> i. Generally less costly construction than deep foundation elements. ii. Rock fill is not required. 	<p>Advantages:</p> <ul style="list-style-type: none"> i. Generally less costly construction than deep foundation elements. ii. Allows construction of footings above the groundwater level. iii. Higher geotechnical resistances compared to footings on native soil. iv. More uniform support than footings partially on native soil. v. Precast concrete footings may be employed 	<p>Advantages:</p> <ul style="list-style-type: none"> i. High geotechnical resistances can be achieved in dense to very dense soils. ii. Installation of piles could continue in freezing weather iii. Excavation below groundwater level may be reduced or eliminated. 	<p>Advantages:</p> <ul style="list-style-type: none"> i. High geotechnical resistances can be achieved in dense to very dense soils. ii. Installation of caissons could continue in freezing weather iii. Excavation below groundwater level may be reduced or eliminated.
<p>Disadvantages:</p> <ul style="list-style-type: none"> i. Subexcavation below the water level is required. ii. Footings must be placed using tremie concrete, which may not be compatible with proposed culvert type. iv. Greater excavation depths would be required for higher resistance values. v. Potential disturbance of subgrade during excavation. 	<p>Disadvantages:</p> <ul style="list-style-type: none"> i. Subexcavation below the water level is required to place rock fill. ii. Additional cost of rock fill placement compared to footings on native soil. iii. Rock fill cannot be compacted under water. iv. Potential disturbance of subgrade during excavation. 	<p>Disadvantages:</p> <ul style="list-style-type: none"> i. Driven piles are expected to encounter refusal on cobbles and boulders at varying depths and often above the bedrock surface. ii. Predrilling through cobbles and boulders will be difficult. iii. Pile lengths will be inadequate at the outlet due to shallow bedrock. Socketing into the bedrock would be required. iii. Higher unit costs than footings. iv. Pile lengths may vary. 	<p>Disadvantages:</p> <ul style="list-style-type: none"> i. Augering and advancement of liner may be obstructed by cobbles and boulders and the very dense nature of the soils at site. ii. Higher cost than spread footings iii. Specialized installation measures such as temporary liners and drilling mud will be required to install caissons in cohesionless soils below the water table. iv. Potential difficulty in cleaning and inspecting bases.
FEASIBLE	RECOMMENDED	NOT RECOMMENDED	NOT RECOMMENDED

Appendix E

List of SPs and OPSS, and Suggested Text for Selected NSSP

1. List of Special Provisions and OPSS Documents Referenced in this Report

- OPSS 206
- OPSS 501
- OPSS 804
- OPSS 902

2 Suggested Text for NSSP on Foundation Excavation

The Contractor is advised that groundwater levels are high at this site and the soils consist of cohesionless sands containing cobbles and boulders. Preparation of the founding surfaces for spread footings will require excavation below the groundwater level within these deposits.

Excavation sidewalls in the cohesionless deposits will generally be unstable and sloughing due to groundwater inflow must be anticipated. The presence of cobbles and boulders is likely to impede driving of sheet piles, and therefore installation of sheet pile shoring, dewatering of the excavation and construction of culvert footings in the dry is expected to be difficult at this site.

In view of the site conditions, preparation of the founding surface is to entail subexcavation in the wet to the specified depths (compact to dense native soils) in short sections of about 2 m length followed by immediate backfilling with rock fill to above the groundwater level, followed by placement of clear stone to the design founding level as per the Contract Drawings.

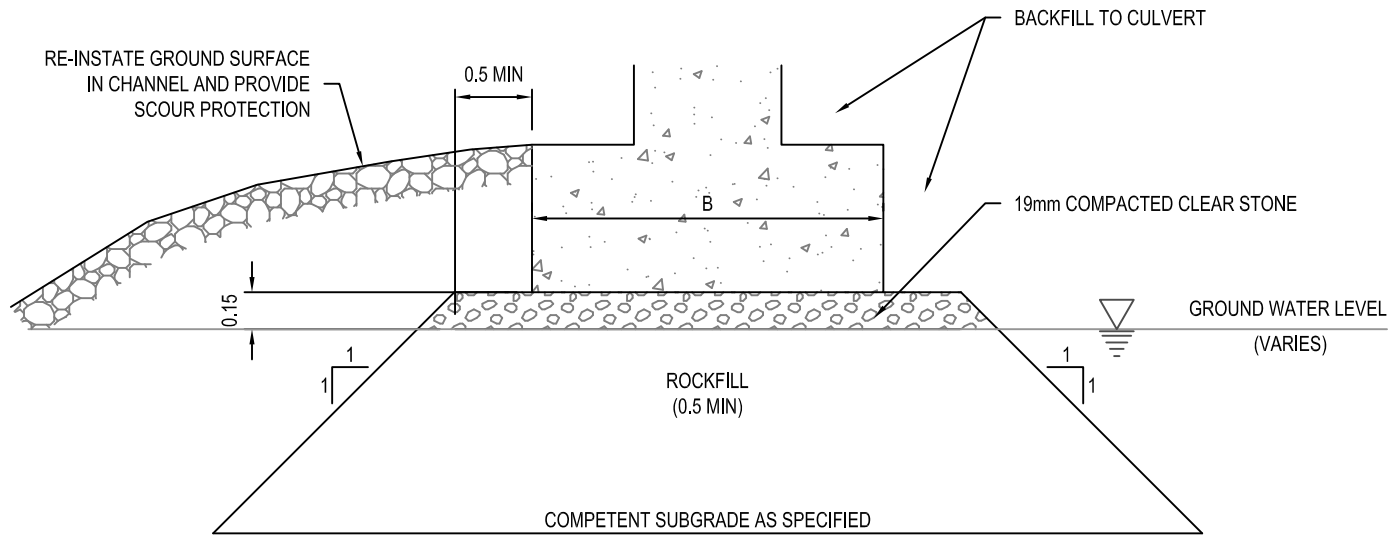
The contractor must carry out the work in a manner which minimizes disturbance to the excavation base and ensure that the excavation does not encroach into the creek by controlling the length of excavation open at any one time, use of shoring, or other suitable means.

Large boulders may be encountered within the excavation depth. Removal of these boulders will require appropriate excavating equipment, and may result in areas of over-excavation requiring additional rock fill to backfill.

Selection of the equipment and methodology to excavate and prepare the founding surface remains the responsibility of the Contractor, and should be based on his interpretation of the subsurface conditions presented in the Foundation Investigation Report as well as the surface conditions exposed at the site.

Appendix F

Figure F1 – Details of Footing on Rock Fill



CROSS-SECTION

NOTES:

1. REMOVE ANY TOPSOIL AND SOFT/LOOSE SUBSOIL UNDER AREA OF ROCKFILL TO COMPETENT SUBGRADE LEVEL AS SPECIFIED.
2. PLACE ROCKFILL TO ABOVE GROUNDWATER LEVEL. ROCKFILL TO HAVE PARTICLE SIZE NO GREATER THAN 150mm.
3. ROCKFILL SURFACE SHOULD BE COMPACTED WITH SEVERAL PASSES OF A DOZER/ROLLER AFTER ROCKFILL IS ABOVE WATER LEVEL.
4. PLACE CLEAR STONE TO BASE OF FOOTING LEVEL AND COMPACT THE CLEAR STONE.
5. PLACE CONCRETE FOOTING.
6. RE-INSTATE GROUND SURFACE IN CHANNEL AND PROVIDE SCOUR PROTECTION.

FOOTING ON ROCKFILL CORE



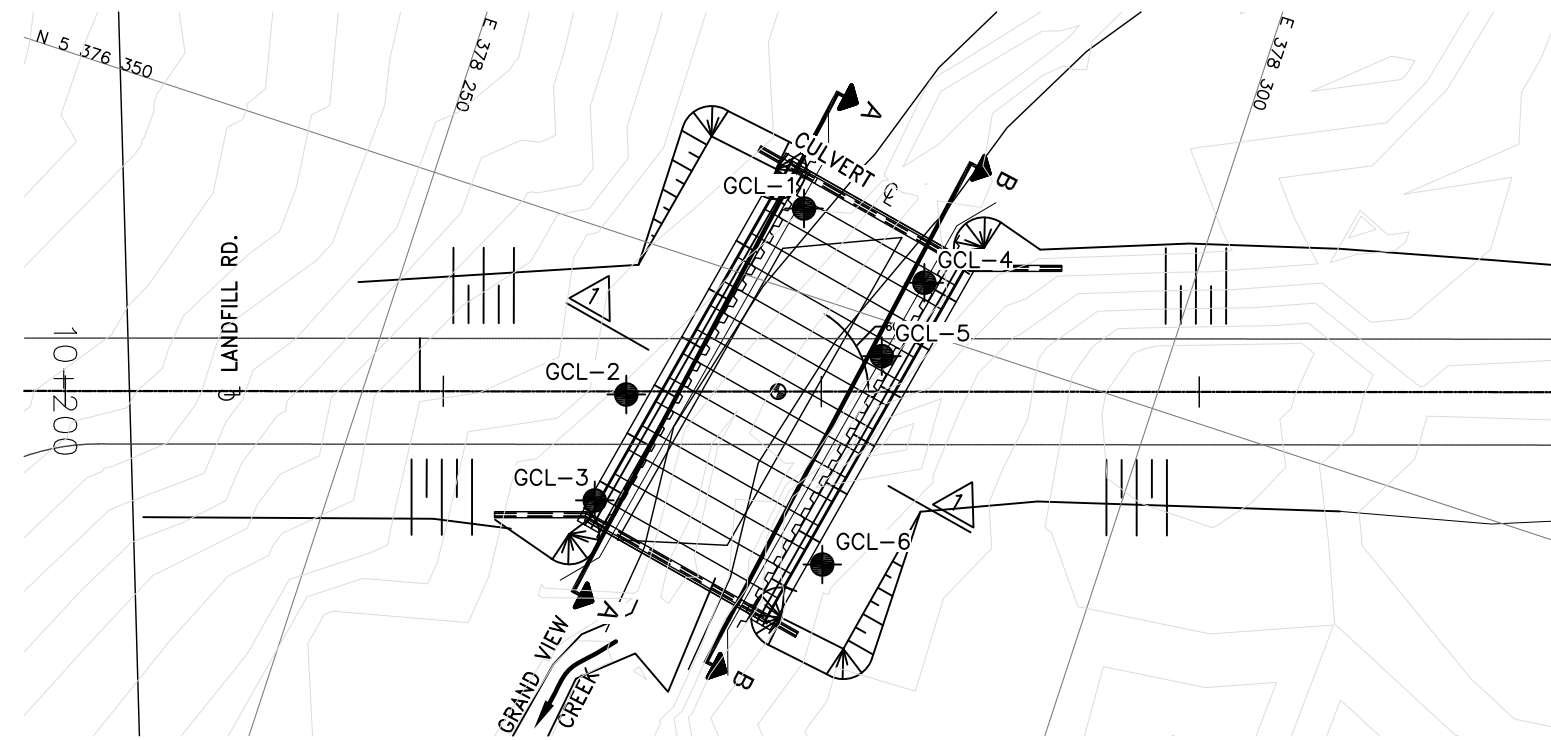
THURBER ENGINEERING LTD.

ENGINEER:	MRA	DRAWN:	MFA	APPROVED:	-
DATE:	SEPTEMBER 2012	SCALE:	N.T.S.	DRAWING No.	FIGURE F1

Appendix G

Borehole Locations and Soil Strata Drawing

METRIC
DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN



CONT No 2012-6010
WP No 623-89-00

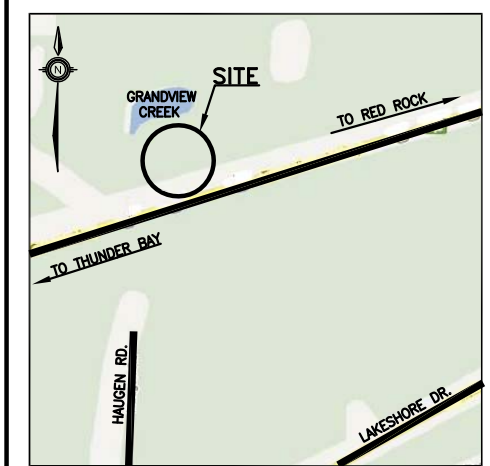
GRANDVIEW CREEK
LANDFILL ROAD
CULVERT REPLACEMENT
BOREHOLE LOCATIONS AND SOIL STRATA



SHEET
264

Hatch Mott MacDonald

THURBER ENGINEERING LTD.



KEYPLAN

LEGEND

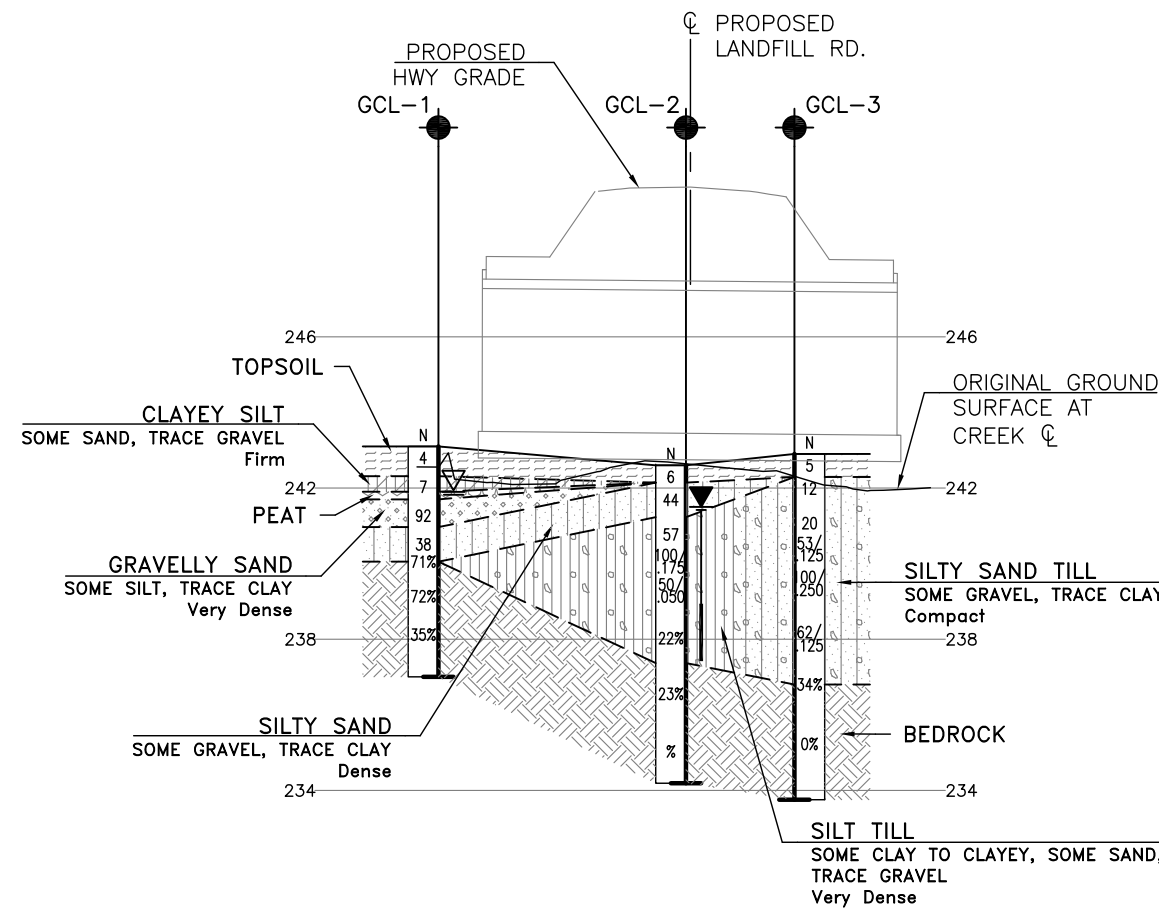
- Borehole
- ⊕ Borehole and Cone
- N Blows /0.3m (Std Pen Test, 475J/blow)
- CONE Blows /0.3m (60° Cone, 475J/blow)
- PH Pressure, Hydraulic
- ▽ Water Level
- ⬆ Head Artesian Water
- ⬆ Piezometer
- 90% Rock Quality Designation (RQD)
- A/R Auger Refusal

NO	ELEVATION	NORTHING	EASTING
GCL-1	243.1	5 376 355.4	378 274.0
GCL-2	242.6	5 376 340.0	378 266.7
GCL-3	242.9	5 376 332.7	378 266.9
GCL-4	242.9	5 376 353.2	378 283.1
GCL-5	242.8	5 376 347.7	378 281.9
GCL-6	243.3	5 376 333.4	378 282.5

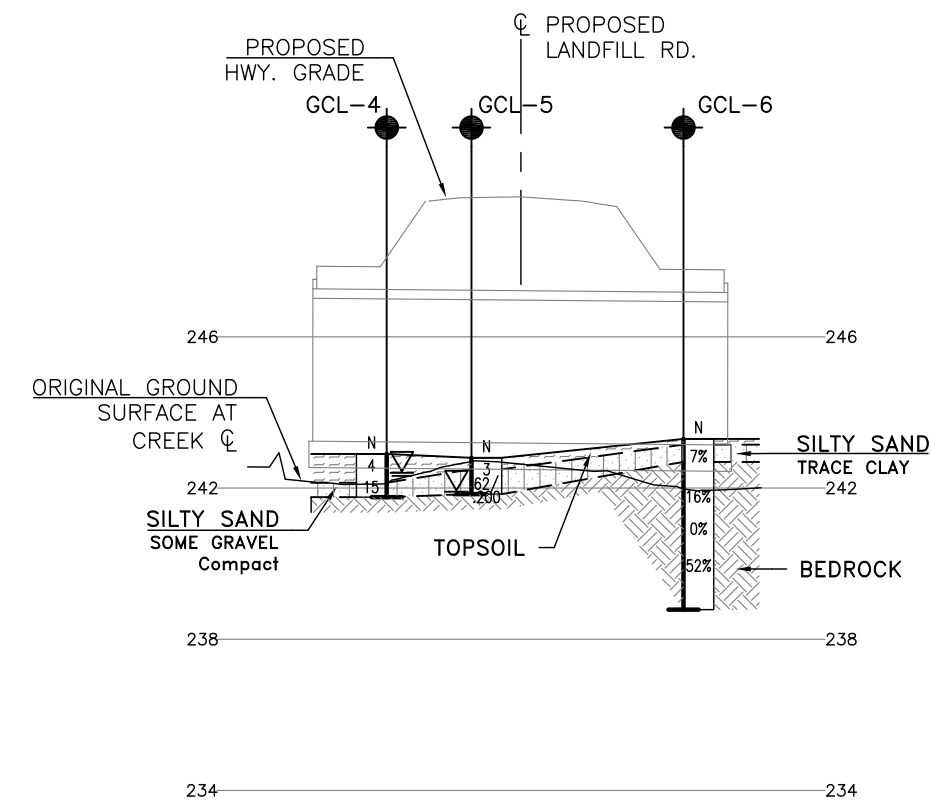
-NOTES-

- The boundaries between soil strata have been established only at Borehole locations. Between Boreholes the boundaries are assumed from geological evidence.
- This drawing is for subsurface information only. Surface details and features are for conceptual illustration.

GEOCRES No. 52A-159



SECTION A-A



SECTION B-B

REVISIONS	DATE	BY	DESCRIPTION
DESIGN	MC	CHK MC	CODE CAN/CSA S6-06 [LOAD CL-625-ONT] DATE JULY 2012
DRAWN	AN	CHK RPR	SITE 48C-351/C [STRUCT] DWG 2