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FOUNDATION INVESTIGATION AND DESIGN REPORT
HEAVEN CREEK CULVERT REPLACEMENT
HIGHWAY 811 (14.9 KM WEST OF HIGHWAY 527)
THUNDER BAY DISTRICT
AGREEMENT NO.: 4014-E-0023
SITE NO.: 48W-197/C
GEOCRES NO.
GWP 6361-14-00

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PART 1: FACTUAL INFORMATION

1. INTRODUCTION

DST Consulting Engineers Inc. (DST) has been retained by the Ministry of Transportation (MTO), Geotechnical Section, Eastern Region to conduct a foundation investigation and produce a foundation design report for the proposed culvert replacement on Highway 811. This work was carried out under Agreement No.: 4014-E-0023. This report addresses the field investigation, laboratory test program, factual report on conditions (Part 1) and recommendations for design and construction for the proposed culvert replacement (Part 2).

2. SITE DESCRIPTION

The site is located on Highway 811, approximately 14.9 km West of Highway 527 (Latitude 49.398897, Longitude -89.565556), Station 10+056, unsurveyed territory, in the District of Thunder Bay.

It is understood that the existing structure at this location is a 3.05 m wide 2.28 m in height x 17.6 m in length Corrugated Steel Plate (CSP) twin culvert with a thickness of soil cover of approximately 0.5 m. The existing culverts (Figures 2.1 and 2.2) inspection by others indicates deformation near outlet of both cells with moderate rusting observed 300 mm above water level. The deformation of soffit in both cell at the inlet and lifting of culvert at the outlet was identified.

The height of the embankment is approximately 1.2 m and the side slope of the embankment is approximately 1.5H: 1V. The surrounding area is moderately vegetated and wooded (Figures 2.1 and 2.2). Photographs were taken by others.

Geological information is available from published *Ontario Geological Survey Map #52HSW* by the *Ontario Ministry of Natural Resources* for the Heaven Lake area. The map indicates that the local area landform is identified as ground moraine. The term "ground moraine", refers to an extensive deposit of till forming an undulating to rolling plain. Although till is generally composed of an assortment of particle sizes (including clay, silt, sand, gravel, cobbles, and boulders), till deposits in this map-area have a high sand and boulder content. The thickness of till in ground moraine varies from less than 1.0 m to many tens of meters. The landform is mapped as ground moraine where the till is thick enough to mask any topographic effect of underlying bedrock. In general, the till layer forms a mantle less than 3 m thick over the bedrock. Till in ground moraine tends to be thicker in bedrock depressions and thinner over bedrock ridges and knobs.

Figure 2.1 Location of existing culvert at Highway 811 (looking South Upstream)

Figure 2.2 Location of existing culvert at Highway 811 (looking North Downstream)

3. INVESTIGATION PROCEDURES AND LABORATORY TESTING

Site work was carried out during September 18th to 22th, 2015 utilizing a CME 750 drill rig equipped for geotechnical drilling and operated by DST. A total of four boreholes were advanced to depths ranging from 10.2 m to 14.5 m. The minimum number and depth of the boreholes was specified by the Ministry of Transportation (MTO).

The borehole locations and stratigraphic sections are shown on the Borehole Location Plan and Drawings 1 and 2 in Appendix C. Borehole 1 was advanced West of the existing West culvert inlet at Station 10+063, 11 m left of centreline, and advanced to a depth of 14.5 m below existing surface. Borehole 2 was advanced West of the existing culvert at Station 10+061, 6.0 m right of centreline, and advanced to a depth of 12.8 m below existing surface. Borehole 3 was advanced at the outlet of the existing East culvert at Station 10+049, 14 m right of centreline, and advanced to a depth of 10.2 m below existing surface. Borehole 4 was advanced at East of the existing culvert at Station 10+051, 6.0 m left of centreline, and advanced to a depth of 13 m below surface.

The borehole locations are referenced to the MTO Station numbering system as indicated on the drawings provided by the Ministry. The ground surface elevations at the borehole locations were surveyed by DST personnel and referenced to benchmark 449.305 m HCP 100 (N = 5473514.644, E = 336371.662) as indicated on the drawings provided by the Ministry. Table 3.1 summarizes the detail of borehole locations and depths.

All boreholes were abandoned using suitable abandonment barrier as described in Ontario Regulation 903 and its amendments. Boreholes were decommissioned by backfilling to the bottom of the road base with cuttings and bentonite chips. From the bottom of the road base, granular materials were replaced to the bottom of the asphalt and the asphalt was sealed with a cold patch.

The fieldwork was supervised on a full-time basis by DST personnel who located the boreholes in the field, performed sampling, in-situ testing and logged the boreholes. Soil samples were obtained from the auger flights and from the split spoon sampler used for the standard penetration test (SPT). The SPT involves driving a 51 mm diameter thick-walled sampler into the soil under the energy of a 63.5 kg weight falling through 760 mm. The number of blows required to drive the sampler 305 mm is known as the standard penetration blow count (N) which provides an indication of the condition or consistency of the soil. The soil samples collected during drilling were identified in the field, placed in labelled containers and transported to DST's laboratory in Thunder Bay for further analyses.

Classification and index tests were subsequently performed in the laboratory on samples collected from the boreholes to aid in the selection of engineering properties. Laboratory tests included moisture contents and particle size analyses. A total of forty five (45) moisture contents, and six (6) particle size analyses have been carried out for this assignment. Laboratory test results are presented in the Boreholes Logs and graphical plots attached Appendix D (Enclosures).

Table 3-1 Detail of Borehole Location

Borehole ID	Station	Elevation (m)	Depth (m)	Offset (m)
BH1	10 + 063	448.0	14.5	11 Lt
BH2	10 + 061	449.0	12.8	6.0 Rt
BH3	10 + 049	447.9	10.2	14 Lt
BH4	10 + 051	449.2	13.0	6.0 Rt

4. DESCRIPTION OF SUBSURFACE CONDITIONS

The subsurface conditions are presented based on the information obtained during power auger drilling.

The generalized stratigraphy of the existing embankment and culvert inlet and outlet, based on the conditions encountered in the boreholes consists of sand fill overlying silt and sand which further overlies till. Topsoil was encountered at the surface in Boreholes 1 and 3 overlying the sand fill. Summary of the soil strata are shown in Table 4-1 and 4-2 below.

Table 4-1: Summary of soil strata at the culvert embankment location (BH2 and BH4)

Layer	Depth (m)	Elevation (m)	Comments
Fill - Sand	0.0 to 3.5 0.0 to 3.8	449.0 to 445.5 449.2 to 445.4	BH2 BH4
Silt and sand	3.5 to 12.8 3.8 to 12.2	445.5 to 436.2 445.4 to 437.0	BH2 BH4
Till	12.2 to 13.0	437.0 to 436.2	BH4

Table 4-2: Summary of soil strata at the culvert inlet and outlet location (BH1 and BH3)

Layer	Depth (m)	Elevation (m)	Comments
Top soil	0.0 to 0.1 0.0 to 0.1	448.0 to 447.9 447.9 to 447.8	BH1 BH3
Fill - Sand	0.1 to 2.7 0.1 to 1.0	447.9 to 445.3 447.8 to 446.9	BH1 BH3
Silt and sand	2.7 to 14.5 1.0 to 9.1	445.3 to 433.5 446.9 to 438.8	BH1 BH3
Till	9.1 to 10.2	438.8 to 437.7	BH3

4.1 Topsoil

Topsoil was encountered at surface in Borehole 1 and 3 with thickness of 100 mm (Elev. 448.0 to 447.9 m and Elev. 447.9 to 447.8 m) respectively.

4.2 Fill – Sand

Sand fill, gravelly to trace gravel with some to trace fines was encountered at surface in Boreholes 2 and 4 and below the topsoil in Boreholes 1 and 3 at depths from 0.0 to 3.5 m (Elev. 449.0 to 445.5 m), 0.0 to 3.8 m (Elev. 449.2 to 445.4 m), 0.1 to 2.7 m (Elev. 447.9 to 445.3 m), and 0.1 to 1.0 m (Elev. 447.8 to 446.9 m) with thicknesses of 3.5 m, 3.8 m, 2.6 m and 0.9 m.

SPT 'N' values vary from 2 to 27, indicating a very loose to compact condition. The moisture contents of the sand material vary from 6 to 31 %. The laboratory test results are summarized in Table 4-3.

Table 4-3 Summary of Sieve Analysis - Fill - Sand

Laboratory Results – Sieve Analysis	
Gravel %	2 to 31
Sand %	60 to 81
Fines %	8 to 19

4.3 Silt and Sand

Silt and sand with trace gravel was encountered below the fill in all four boreholes at depths from 2.7 to 14.5 m (Elev. 445.3 to 433.5 m), 3.5 to 12.8 m (Elev. 445.5 to 436.2 m), 1.0 to 9.1 m (Elev. 446.9 to 438.8 m), and 3.8 to 12.2 m, (Elev. 445.4 to 437.0 m) with thicknesses of 11.8 m, 9.3 m, 8.1 m, and 8.4 m.

SPT 'N' values vary from 6 to 33, indicating a loose to dense condition. The moisture contents of the sand material vary from 15 to 32 %. The laboratory test results are summarized in Table 4-4.

Table 4-4 Summary of Sieve Analysis- Silt and Sand

Laboratory Results – Sieve Analysis	
Gravel %	3
Sand %	46
Fines %	51

4.4 Till

Sand till was encountered below the silt in Boreholes 3 and 4 at depths from 9.1 to 10.2 m (Elev. 438.8 to 437.7 m) and 12.2 to 13.0 m (Elev. 437.0 to 436.2 m) with thicknesses of 1.1 m and 0.8 m.

SPT 'N' values vary from 55 to 100+, indicating a very dense condition. The moisture contents of the sand material vary from 8 to 18 %. The laboratory test results are summarized in Table 4-5.

Table 4-5: Summary of Sieve Analysis- Till – Sand

Laboratory Results – Sieve Analysis	
Gravel %	19
Sand %	42
Fines %	39

4.5 Dynamic Cone Testing

DCPT was advanced in Borehole 1 at the depth from 10.0 to 14.5 m (Elev. 438.0 to 433.5 m). DCPT values obtained range from 11 to 53 blows per 0.3 m penetration respectively.

4.6 Groundwater

At the time of the field investigation groundwater was observed in Boreholes 1, 3, and 4. The groundwater levels can be expected to vary with the season and precipitation events.

Table 4-6: Groundwater depth

Borehole	Groundwater Depth (m)	Groundwater Elev. (m)
Borehole 1	5.0	443.0
Borehole 3	1.1	446.8
Borehole 4	2.1	447.1

5. MISCELLANEOUS

Site work was carried out during the week of September 18th to 22th, 2015 utilizing a CME 750 all-terrain drill rig operated by DST personnel. Fieldwork was supervised on a full time basis by Peter Raynak who located the boreholes in the field, performed sampling, in-situ testing and logged the boreholes. Soil samples collected during drilling were identified in the field, placed in labelled containers and transported to DST's laboratory in Thunder Bay for further analysis. Interpretation of the data and preparation of the report was completed by Selorm Danku, P.Eng and reviewed by Prof. Myint Win Bo, P.Eng a designated principal contact for MTO projects.

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PART 2: ENGINEERING DISCUSSIONS AND RECOMMENDATIONS

6. PROJECT DESCRIPTION

DST Consulting Engineers Inc. has been retained by the Ministry of Transportation (MTO), Geotechnical Section, Eastern Region, to conduct a foundation investigation and design report for the proposed culvert replacement at Highway 811. This work was carried out under Agreement No.: 4014-E-0023.

Existing culvert structure at this location is 3.1 m wide x 2.28 m high x 17.6 m in length, Corrugated Steel Plate (CSP) twin culvert with a thickness of soil cover of approximately 0.5 m. The existing culverts inspection by other indicates deformation near outlet of both cells with moderate rusting observed 300 mm above water level. The deformation of soffit in both cell at the inlet and lifting of culvert at the outlet was identified.

The embankment at the culvert location is approximately 2.7 to 3.8 m high. The surrounding area is moderately vegetated.

Generalized stratigraphy of the existing embankment, based on the conditions encountered in boreholes, consists of fill sand underlain by silt and sand which underlain again by silty sand (Till).

This section presents interpretation of the geotechnical data presented in the factual report and provides geotechnical design recommendations and construction concerns for the proposed culvert replacement.

6.1 Replacement Structure

It is understood that three options (Box culvert, Precast Concrete Open Footing Culvert, and Corrugated Steel Plate Culvert) have been discussed as a replacement structure. It is also understood that open cut excavation will be used to replace the structure.

The design of the replacement structure must be in accordance with the Canadian Highway Bridge Design Code CAN/CSA-S6-06 (CHBDC, 2006) and all relevant Ministry of Transportation specifications and guidelines.

6.2 Foundation Design

The culverts will be located at the same vertical and horizontal alignment of the existing culvert structure. As the proposed culverts are not expected to be heavily loaded, a shallow foundation is considered suitable for this site. As the cross sectional area of the replacement structure is expected to be larger than the existing culvert, the overall effect on the culvert foundation soils by the embankment load will be a decrease in stress at the base of the culvert.

If sub-excavation for frost effects is carried out in the dry (with adequate dewatering controls), the material can be replaced with Granular A material meeting OPSS.PROV 1010 specifications and compacted to a minimum of 95 % of standard Proctor maximum dry density in accordance with OPSS.PROV 501 “Construction Specification for Compacting”. If sub-excavation for frost effects is carried out in the wet (water is maintained at or above adjacent groundwater table) All foundation preparation should be completed in accordance with OPSS 421 “Construction Specification for Pipe Culvert Installation in Open Cut”, any specifications provided in the contract documents and as indicated in Section 6.7 Bedding.

6.2.1 Foundation Design (Box culvert)

The geotechnical resistance was estimated for the ultimate limit state (ULS) and serviceability limit state (SLS) for a maximum settlement of 25 mm. The resistance at ULS was calculated by applying load resistance factor of 0.5 in accordance with the Bridge Design Code (CHBDC) CAN/CSA-S6-06 section 6.6.3.6, and is shown in Table 6-1.

While ULS is not relevant at final condition of box culvert due to significant soil cover, SLS is not relevant for temporary condition. Therefore SLS reported here are for final condition. The recommended bearing resistance of at SLS is in reference to an allowable net increase over the existing loading condition.

The geotechnical resistance was estimated assuming a strip footing consisting of a width equal to the width of the box culvert (9.9 m), length of 18.3 m and a depth of the culvert base equal to 0 m, which is a temporary condition prior to backfill that will be encountered during construction. The culvert can be installed below the pavement and top of bedding material placed on undisturbed native soils.

Table 6-1 Geotechnical resistances and reactions

Footing Size	Ultimate bearing capacity (kPa)	Resistance at ULS (kPa)	Resistance at SLS (kPa)
B = 9.9m	580	290	35

Bearing resistance for SLS was found to be significantly lower than the resistance for ULS. However, the soil encountered is sandy in nature and immediate settlement is expected. Therefore, culvert could be designed with ULS resistance provided that significant culvert settlement will be completed during the construction time.

Where unsuitable or unstable soils are encountered, such as soil mixed with organic, the foundation soils must be removed to a firm or hard soils and replaced to the foundation grade with Granular “A” material meeting OPSS.PROV 1010 specifications and compacted to a minimum of 95 % of standard Proctor maximum dry density.

6.2.2 Foundation Design (Precast Concrete Open Footing and Corrugate Steel Plate Culvert)

As the cross sectional area of the proposed culvert will remove the existing soil materials, the overall effect on the culvert foundation soils will be a small decrease in stress at the base of the culvert.

The geotechnical resistance was estimated for the ultimate limit state (ULS) and serviceability limit state (SLS) for a maximum settlement of 25 mm. The resistance at ULS was calculated by applying load resistance factor of 0.5 according to the Bridge Design Code (CHBDC) CAN/CSA-S6-06 section 6.6.3.6, and is shown in Table 6-2. The geotechnical resistance was estimated assuming a strip footing of various widths with a length equal to 18.3 m situated at depth approximately 3.9 m below the existing road elevation.

Table 6-2 Geotechnical resistances and reactions for open footing culverts

Footing Width L=18.3 m	Depth of Soil Cover	Ultimate bearing capacity (kPa)	Factored Resistance at ULS (kPa)	Resistance at SLS (kPa)
B = 1.0 m	1.0	250	125	125
	1.2	290	145	125
	1.5	340	170	125
B = 1.2 m	1.0	270	135	110
	1.2	300	150	110
	1.5	360	180	110
B = 1.5 m	1.0	290	145	95
	1.2	330	165	95
	1.5	380	190	95
B = 2.2 m	1.0	340	170	75
	1.2	380	190	75
	1.5	440	220	75
B = 3.0 m	1.0	400	200	60
	1.2	440	220	60
	1.5	500	250	60

Bearing resistance for SLS was found to be significantly lower than the resistance for ULS. However, the soil encountered is sandy in nature and immediate settlement is expected. Therefore, culvert could be designed with ULS resistance provided that significant culvert settlement will be completed during the construction time.

Culvert foundation can be constructed either with dewatering during excavation or without lowering the water in the excavation. If the construction of foundation is carried out in the wet, foundation construction can be performed by Tremie concrete placement. In this case, soil below foundation should be carefully prepared to minimize the disturbance. Alternatively, use of precast concrete footings can be considered.

6.3 Lateral and Sliding Resistances

The analysis of horizontal and vertical effects of earth loads on the culvert can be performed considering soil parameters given in Table 6-3 and as described in Section 7.6.3.1 in Canadian Highway Bridge Design Code. Temporary bracing and shoring may be designed using the typical soil parameters given in Table 6-3 and Table 6-4, however the designer/contractor should verify the appropriate soil parameters for the designs of specific bracing and shoring system.

It is recommended that all excavations be either adequately sloped or securely shored and braced to prevent earth caving and to provide a safe and stable work area. The design should incorporate the effects of hydrostatic pressure, traffic surcharge and retained sloping earth conditions in the bracing design.

Table 6-3 Typical Soil Parameters for Earth Loads

Soil type	Unit weight (kN/m^3)	Internal drained friction angle (Deg)	Interface friction angle δ (Deg)
Fill Sand	20	30	20
Silt & Sand	20	30	20
Silty Sand (Till)	22	32	21

Table 6-4 Lateral Earth Pressure Coefficients

Soil type	Active Earth Pressure (K_a)	Passive Earth Pressure, (K_p)	Earth Pressure at Rest, (K_0)
Equation *	$\left(\frac{1 - Sin\emptyset}{1 + Sin\emptyset}\right)$	$\left(\frac{1 + Sin\emptyset}{1 - Sin\emptyset}\right)$	$(1 - Sin\emptyset)$
Fill Sand	0.33	3.00	0.50
Silt & Sand	0.33	3.00	0.50
Silty Sand (Till)	0.30	3.25	0.47

* Φ is an angle of internal friction
**The earth pressure coefficients provided here are for the normally consolidated soils condition considering fully mobilized condition

For over consolidated (OC) soils, the earth pressure coefficient at rest condition should be corrected using a following relationship;

$$\mathbf{K_{0\,(OC)} = K_{0(NC)} * (OCR)^{0.5}}$$

Where

$K_{o(oc)}$)= Earth pressure coefficient over consolidated soils

$K_{o(Nc)=}$ Earth pressure coefficient normally consolidated soils

OCR= Over Consolidation Ratio

The sliding resistance can be calculated using the following formulae.

$$F_r = W (\tan\delta)$$

Where,

δ = Interface friction angle

W= Total weight of the soil element retained per unit length of the retaining wall

6.4 Staged Construction

Staged construction method is being considered to complete the culvert replacement, with vertical shoring and minor road widening with side-slopes at the excavation. The staged construction includes two (2) stages.

The method of road widening with side-slopes 1.5H: 1V at the excavation for culvert replacement is a staged construction and road widening to provide a single lane of traffic without a detour. The staged construction includes two (2) stages, Stage 1 and Stage 2 is a temporary lane diversion which involves temporary detour of traffic to the single lane.

Slope stability analyses indicate that factor of safety for excavation depth of 3.5 m from the road surface with 1.5H: 1.0V is 1.3 (Drained). The embankment slopes at the inlet and outlet should be 2H: 1V. Embankment with side-slopes 1.5H: 1V at the excavation for culvert replacement is feasible with a factor of safety of 1.3.

The vertical shoring method for culvert replacement is staged construction to provide a single lane of traffic without a detour, Stage 1 and Stage 2 is a temporary lane diversion which involves temporary detour of traffic to the single lane and as well as the installation of level II roadway protection. Use of temporary concrete barriers or steel sheet piles will be required. The final embankment foreslopes should be reinstated as presented in Section 6.12 Embankment Foreslopes. The proposed shoring system may require predrilling in those locations with high concentration of boulders and cobbles and tiebacks at different elevations in the excavation.

6.5 Earth Excavation

Earth excavation will be required adjacent to the existing and replacement structure and may require temporary surface water ditch diversion and temporary support for traffic. This method can more readily accommodate excavation of large boulders, if encountered during excavation. As a minimum, the procedures should be in accordance with OPSS 902 “Construction Specifications for Excavating and Backfilling-Structures”. Where temporary protection systems are required they shall be constructed in accordance with OPSS.PROV 539 “Construction Specification for Temporary Protection Systems” and Section 6.6 “Roadway Protection”.

If organic materials are encountered during excavation, the excavations to remove these organics and wood should be completed in accordance with OPSS.PROV 209 “Construction Specification for Embankments Over Swamps and Compressible Soils”.

Open excavation without shoring could be completed provided that the soils are sloped back sufficiently to maintain sidewall stability and protect workers. The stability of the excavation side slopes will be highly dependent on the contractor’s methodology and ability to effectively dewater the excavation. Bottom width of excavation should be 4 to 6 m wider than maximum width of proposed replacement culvert.

6.6 Roadway Protection

Roadway protection for this project should be constructed in accordance with the requirements of the Occupational Health and Safety Act of Ontario (OHSA), O.Reg. 213/91. According to O.Reg. 213/91, s.226, the soils in the area of interest is classified as Type 3. Type 3 soils generally are stiff to firm and compact to loose or are previously excavated soil, exhibit signs of surface cracking, exhibit signs of seepage, if it is dry, may run easily into a conical pile and have a low degree of internal strength. In accordance with O. Reg. 213/91, s.227 (3), if an excavation contains more than one type of soil, the soil shall be classified with the highest number as described in section 226. These should be assessed and confirmed in the field as construction progresses.

Since temporary roadway protection is required during the structure replacement, installation of a sheet pile or soldier pile wall may be considered to ensure the stability of the bank and is a feasible option. The design of roadway protection may be performed using the typical soil parameters given in Table 6-3 and Table 6-4, however the designer/contractor should verify the appropriate soil parameters for the designs. As the potential of encountering concrete, rock fill, cobbles and boulders exists, the contractor should be prepared to handle this with the selection of adequate driving or vibratory equipment as well as steel thickness. The construction methodology must be in accordance with all applicable standards and regulations related to the method proposed. The contractor’s method and equipment must be suitable for the site conditions and materials used. This soil investigation does show evidence of shallow bedrock, cobbles and boulders along the soil profile, this may create a construction constrain to install any driven retaining wall.

6.7 Bedding

The foundation soils, silts in particular, will be very susceptible to disturbance and weakening as a result of traffic, standing water and frost. Any foundation soils that could be disturbed shall be protected. The bottom of the excavation on which the culvert or granular pad is to rest shall not be disturbed. The bedding placement should commence immediately after the final removal of material to the foundation level has been completed.

The bedding for the structure should be designed in accordance with Section 7.8 of the CHBDC. The bedding shall be a minimum of 0.5 m thick and extend to a minimum width (half of the width of culvert) beyond all sides of the culvert. The bedding material should consist of “Granular A” as per Soil Group I in accordance with Table 7.4 of the Canadian Highway Bridge Design Code. The “Granular A” shall be in accordance to OPSS.PROV 1010. The “Granular A” should be placed in layers not exceeding 200 mm in thickness, loose measurement, and each layer compacted to a minimum of 95 % of standard Proctor maximum dry density. The middle one-third of the culvert width of the top bedding layer, having minimum thickness of 75 mm, shall be loosely placed and uncompacted.

If construction is performed without dewatering, bedding material should consist of 19 mm Type I or II clear stone as defined in OPSS.PROV 1004.05.02. If fine materials are present beneath the clear stone a non-woven geotextile (OPSS 1860.07.05.01 Class II) with the filtration opening size (FOS) less than 135 µm may be required for separation. No compaction is required of the clear stone.

6.8 Sidefill and Overfill

The material used for culvert sidefill should not contain debris, organic matter, frozen materials, or large stones of a diameter greater than one-half the thickness of the compacted layers being placed or 100 mm, whichever is smaller. Soils shall be deposited uniformly on each side of the structure in order to prevent lateral displacement. The minimum width of the sidefill should be at least half of the culvert width on each side. The sidefill should consist of Granular A” and compacted to 95% of standard Proctor maximum dry density.

Overfill should consist of “Granular A” and should be compacted to not greater than the compaction or equivalent stiffness of soils in the sidefill zone and bedding. The backfill materials should be separated from the adjacent soil with a non-woven Class II geotextile, with a filtration opening size of between 50 and 100 µm, specified in OPSS 1860 “Material Specifications for

Geotextiles".

6.9 Channel Diversion and Dewatering

The culvert shall be replaced by diverting the creek channel temporarily adjacent to the existing culvert. It is important to ensure that a flood in the channel does not cause damage to the partly constructed permanent works, to the temporary works or to plant. Floods have a habit of occurring overnight or at weekends and inadequate temporary works can fail with expensive consequences.

If the creek has comparatively a small amount of flow that may depend on the season, it may be feasible for the creek flow to be directed by staging construction. In order to prevent back up of water from upstream and downstream, a dyke made of sand bags has sometimes been used as a hydraulic barrier. However, a sheet pile vertical cut-off wall will provide better control of both surface and groundwater. A suitable sump and pump system, possibly supported by an efficient well-point system, will be required to dewater and stabilize the excavation. A well designed well-point system with a suitable diameter of well point at an appropriate spacing will perform better for working under dry condition and to prevent disturbance of the excavation base through sand boiling and hydraulic heave. It should be noted that depending on the season, depth of excavation and amount of water flow through the creek may vary. The contractor should be prepared to tackle this situation. The contractor should be alerted of the high water table and surface water, for example through a non-standard special provision (NSSP).

A continuous dewatering operation must be provided to keep the excavation stable and free of water. The excavation must be monitored daily throughout the duration of excavation until the completion of backfilling to confirm this. The dewatering system must be maintained and the surrounding area monitored for impacts to items such as, but not limited to, settlement and groundwater usage. The control of water from the dewatering operation should be accordance with OPSS 518 "Construction Specification for Control of Water from Dewatering Operations".

6.10 Erosion Control

Erosion control is essential at inlet and outlet for the successful performance of a culvert. Generally, rip-rap is used to avoid the erosion at inlet and outlet of the culvert. The rip-rap slows down the flow close to the channel bed and prevents culvert failure by the undermining.

To prevent erosion of the surrounding soils at the inlet, rip-rap Treatment shall be applied in accordance with OPSD 810.020 "Rip-Rap Treatment for Ditch Inlets" and OPSS 511 "Construction

Specification for Rip-Rap, Rock Protection, and Granular Sheeting".

The outlet shall be rip-rapped to prevent erosion of the surrounding soils in accordance with OPSD 810.010 "Rip-Rap treatment for Sewer and Culvert Outlets" and OPSS 511 "Construction Specification for Rip-Rap, Rock Protection, and Granular Sheeting".

To prevent undermining of the bedding (if culvert option is used), cutoff walls shall be installed along the entrance and exit end bottom sides of culvert. Cutoff walls should be designed based on velocity of the water flow and the type of soil underneath.

The temporary erosion and sedimentation measures during the construction of culvert shall be controlled as described in OPSS 805 "Construction Specification for Temporary Erosion and Sedimentation Control Measures".

6.11 Frost Protection

In accordance with OPSD 3090.100 "Foundation Frost Depths for Northern Ontario", the frost penetration at this location for silty soil is about 2.3 m. The frost susceptible soils shall not be used adjacent to the culvert wall within the depth of frost penetration from the road surface. The soils under the culvert are highly frost susceptible (capable of forming thick ice lenses with the associated pressures and heave).

During winter season, ice may form inside the culvert and a low flow rate may assist the ice formation. It is expected that ice may extend to the culvert invert and frost could therefore extend into the soils below the culverts, possibly as deep as 2.3 m. The frost heave may generate additional stresses on the culvert foundation and walls.

Three design approaches are commonly applied; designing the culvert with enough strength and rigidity to tolerate these pressures (recognizing that the maximum differential pressures and movements as a result of frost lensing cannot be accurately quantified); removing the frost susceptible soils within the frost zone; or providing adequate insulation to reduce frost penetration. As the frost penetration is extended below the invert level of the culvert, the frost protection should be in accordance with OPSD 803.030 and 803.031 "Frost Treatment - Pipe Culverts, Frost Penetration Line below Bedding Grade" and "Frost Treatment - Pipe Culverts, Frost Penetration Line between Top of the Pipe and Bedding Grade".

If sub-excavation for frost effects is carried out in the dry (with adequate dewatering controls), the material can be replaced with Granular B Type 1 material compacted to 95% of standard proctor maximum dry density. If the excavation is in the wet (water is maintained at or above adjacent groundwater table) then the material should be rockfill or clear stone surrounded by geotextile, without the need for compaction. Depending on the structural design of the culvert, partial sub-excavation (less than 2.0 m) may also be considered to reduce differential stresses associated with frost; however the exact pressures and movements cannot be accurately quantified.

Acceptable insulation to prevent frost penetration would be 152 mm Dow Styrofoam Highload 40 Insulation or an equivalent material with a compressive strength of approximately 275 kPa or greater. For a region that has a freezing index greater than 1500 Celsius Degree-Days it is recommended that the insulation be placed beneath the structure and extend 2.44 m from the face of the buried structure.

6.12 Embankment Foreslopes

The foreslopes should be reinstated with a slope not steeper than 2H: 1V if being constructed with granular materials. The foreslopes should be reinstated with a slope not steeper than 1.5H: 1V if being constructed with rock fill. The minimum thickness of rock fill must be greater than 2 m to achieve an adequate FOS for the reinstated rock fill embankment.

6.13 Construction Concerns

The main construction issues those need to be addressed for this site are removal of cover/embankment materials, staged removal of the existing culvert, provisions required for temporary roadway protection, diversion of the channel, excavation below the water table and reinstatement of the embankment fill. These items are important for the successful installation of the new culvert.

A quality verification Engineer shall be required to inspect the condition of the foundation and surrounding soils before installation of bedding and other backfills and ensure the width of trench and trench wall slopes are suitable, and ensure compliance with material placements and compaction methods.

7. CLOSURE

Table 7-1 summarizes the advantages and disadvantages of the use of concrete or steel sheet piles wall and 1.5H: 1V sideslopes for roadway protection. Since it is a temporary roadway protection and factor of safety of 1.3 can be achieved, 1.5H: 1V sideslopes is considered to be a recommended option however design of roadway protection is the responsibility of the contractor as per the contract drawings.

Table 7-1 Advantages and Disadvantages of Roadway Protection Methods

Roadway Protection Option	Advantages	Disadvantages
Concrete or steel sheet pile wall	• Relatively non permeable. • Increase erosion control capacity. • Ease installation when working below ground water table. • Can design with suitable factor of safety.	• Difficult driving through cobbles, rock fill. • Higher installation cost. • Special construction equipment and design required.
Side-slope 1.5H:1V	• Does not require specialized equipment. • Relatively short construction time. • Low construction cost. • Can achieve suitable factor of safety.	• Permeable. • Increased erosion due to exposed material. • Low factor of safety with increasing excavation depth.

Table 7-2 Advantages and Disadvantages of the Proposed Culvert Options

Foundation Option	Feasibility	Advantages	Disadvantages	Relative Costs	Risks/Consequences
Option 1- Concrete Box	Feasible	No bearing capacity concerns. Use of pre-cast members could reduce construction time. Low maintenance cost.	Requires roadway protection system. Low SLS.	Low to moderate cost.	In general terms - low risk option. (except for shoring)
Option 2- Precast Concrete Open Footing Culvert	Feasible	Use of pre-cast members could reduce construction time. Low maintenance cost	Use of this option could increase construction time. Requires foundation excavation and preparation. Requires roadway protection system.	Moderate cost.	In general terms low risk option (except for shoring).
Option 3- Corrugated Steel Plate on Open Footing Culvert	Feasible	Use of pre-cast members could reduce construction time. Low maintenance cost.	Use of this option could increase construction time. Requires foundation excavation and preparation. Requires roadway protection system.	Moderate cost.	In general terms low risk option (except for shoring).

8. REFERENCES

Canadian Foundation Engineering Manual. 2006. Fourth Edition, Canadian Geotechnical Society.

Canadian Highway Bridge Design Code. 2006, CAN/CSA-S6-06, A National Standard of Canada, Canadian standards Association.

Municipal and Provincial Common, Volume 1 - General & Construction Specifications, "*Ontario Provincial Standard for Roads & Public Work*s" Spec No. *OPSS* 511, 517, 518, 805, 902.

Municipal and Provincial Common, Volume 2 - Material Specifications, "*Ontario Provincial Standard for Roads & Public Work*s" Spec No. *OPSS* 1860.

Municipal and Provincial Common, Volume 3 - Drawings for Roads, Barriers, Drainage, Sanitary Sewers, Watermains and Structures, "*Ontario Provincial Standard for Roads & Public Work*s" Spec No. *OPSD* 203.040, 803.010, 803.030, 803.031, 810.010, 810.020, 3090.100.

Occupational Health and Safety Act and Regulation, June 2002, Revised Statutes of Ontario, 1990, Chapter O.1, O.Reg. 213/91.

Provincial-Orientated, Volume 5 - MTO General Conditions of Contract and General & Construction Specifications, "*Ontario Provincial Standard for Roads & Public Work*s" Spec No. *OPSS.PROV* 209, 501, 510, 539.

Provincial-Orientated, Volume 6 - Material Specifications, "*Ontario Provincial Standard for Roads & Public Work*s" Spec No. *OPSS.PROV* 1004, 1010.

The Surveys and Design Office, Highway Engineering Division, Ministry of Transportation, 1990, Pavement Design and Rehabilitation Manual.

9. LIMITATIONS OF REPORT

A description of limitations which are inherent in carrying out site investigation studies is given in Appendix ‘A’, and this forms an integral part of this report.

For DST CONSULTING ENGINEERS INC.

Prepared by:	Reviewed by:
Selorm Danku, P.Eng. Geotechnical Engineer	Bernardo Villegas, M.Sc Regional Manager – Technical Services

Reviewed By:

Dr. M W Bo, PhD., P. Eng, P.Geo, Int PE,
C.Geol, C. Eng, Eur Geol, Eur Eng
Senior Vice President / Senior Principal

APPENDIX 'A'

LIMITATIONS OF REPORT

L I M I T A T I O N S   O F   R E P O R T

GEOTECHNICAL STUDIES

The data, conclusions and recommendations which are presented in this report, and the quality thereof, are based on a scope of work authorized by the Client. Note that no scope of work, no matter how exhaustive, can identify all conditions below ground. Subsurface and groundwater conditions between and beyond the testholes may differ from those encountered at the specific locations tested, and conditions may become apparent during construction which were not detected and could not be anticipated at the time of the site investigation. Conditions can also change with time. It is recommended practice that DST Consulting Engineers be retained during construction to confirm that the subsurface conditions throughout the site do not deviate materially from those encountered in the testholes. The benchmark and elevations used in this report are primarily to establish relative elevation differences between the testhole locations and should not be used for other purposes, such as grading, excavation, planning, development, etc.

The design recommendations given in this report are applicable only to the project described in the text and then only if constructed substantially in accordance with details stated in this report. Since all details of the design may not be known, we recommend that we be retained during the final stage to verify that the design is consistent with our recommendations, and that assumptions made in our analysis are valid.

Unless otherwise noted, the information contained herein in no way reflects on environmental aspects of either the site or the subsurface conditions.

The comments given in this report on potential construction problems and possible methods are intended only for the guidance of the designer. The number of testholes may not be sufficient to determine all the factors that may affect construction methods and costs, e.g. the thickness of surficial topsoil or fill layers may vary markedly and unpredictably. The contractors bidding on this project or undertaking the construction should, therefore, make their own interpretation of the factual information presented and draw their own conclusion as to how the subsurface conditions may affect their work.

Any results from an analytical laboratory or other subcontractor reported herein have been carried out by others, and DST Consulting Engineers Inc. cannot warranty their accuracy. Similarly, DST cannot warranty the accuracy of information supplied by the client.

Appendix B

DESCRIPTION OF TERMS

EXPLANATION OF TERMS USED IN REPORT

SPT 'N' VALUE: THE STANDARD PENETRATION TEST (SPT) N VALUE OF THE NUMBER OF BLOWS REQUIRED TO CAUSE A STANDARD 51 mm O.D. SPLIT BARREL SAMPLES TO PENETRATE 0.3 m INTO UNDISTURBED GROUND IN A BOREHOLE WHEN DRIVEN BY A HAMMER WITH A MASS OF 63.5 kg, FALLING FREELY A DISTANCE OF 0.76 m. FOR PENETRATION OF LESS THAN 0.3 m N VALUES ARE INDICATED AS THE NUMBER OF BLOWS FOR THE PENETRATION ACHIEVED. AVERAGE N VALUE IS DENOTED THUS Ñ.

DYNAMIC CONE PENETRATION TEST (DCPT): CONTINUOUS PENETRATION OF A CONICAL STEEL POINT (51 mm O.D. 60° CONE ANGLE) DRIVEN BY 475 J IMPACT ENERGY ON 'A' SIZE DRILL RODS. THE RESISTANCE TO CONE PENETRATION IS MEASURED AS THE NUMBER OF BLOWS FOR EACH 0.3 m ADVANCE OF THE CONICAL POINT INTO THE UNDISTURBED GROUND.

SOILS ARE DESCRIBED BY THEIR COMPOSITION AND CONSISTENCY OR DENSENESS

TEXTURAL CLASSIFICATION OF SOILS

BOULDERS	COBBLES	GRAVEL	SAND	SILT	CLAY
GREATER THAN 200 mm	75 TO 200 mm	4.75 TO 75 mm	0.075 TO 4.75 mm	0.002 TO 0.075 mm	LESS THAN 0.002 mm

COARSE GRAIN SOIL DESCRIPTION (50% GREATER THAN 0.075 mm)

TERMINOLOGY	TRACE OR OCCASIONAL	SOME	WITH	ADJECTIVE (e.g. SILTY OR SANDY)	AND (e.g. SAND AND SILT)
	LESS THAN 10%	10 TO 20%	20 TO 30%	30 TO 40%	40 TO 60%

CONSISTENCY*: COHESIVE SOILS ARE DESCRIBED ON THE BASIS OF THEIR UNDRAINED SHEAR STRENGTH (C_U) AND SPT 'N' VALUES AS FOLLOWS

C_U (kPa)	0 – 12	12 – 25	25 – 50	50 - 100	100 - 200	> 200
N (BLOWS / 0.3 m)	<2	2 - 4	4 - 8	8 - 15	15 - 30	>30
	VERY SOFT	SOFT	FIRM	STIFF	VERY STIFF	HARD

DENSENESS: COHESIONLESS SOILS ARE DESCRIBED ON THE BASIS ON DENSENESS AS INDICATED BY SPT 'N' VALUES AS FOLLOWS

N (BLOWS / 0.3 m)	0 – 5	5 – 10	10 – 30	30 – 50	> 50
	VERY LOOSE	LOOSE	COMPACT	DENSE	VERY DENSE

ROCKS ARE DESCRIBED BY THEIR COMPOSITION AND STRUCTURAL FEATURES AND/OR STRENGTH

RECOVERY: SUM OF ALL RECOVERED ROCK CORE PIECES FROM A CORING RUN EXPRESSED AS A PERCENT OF THE TOTAL LENGTH OF THE CORING RUN

MODIFIED RECOVERY: SUM OF THOSE INTACT CORE PIECES, 100 mm+ IN LENGTH EXPRESSED AS A PERCENTAGE OF THE LENGTH OF THE CORING RUN.

THE ***ROCK QUALITY DESIGNATION*** *(R.Q.D)* FOR MODIFIED RECOVERY IS:

R.Q.D (%)	0 – 25	25 – 50	50 – 75	75 – 90	90 – 100
	VERY POOR	POOR	FAIR	GOOD	EXCELLENT

LEGEND OF RECORDS FOR BOREHOLES: SYMBOLS AND ABBREVATIONS FOR SAMPLE TYPE

SS	SPLIT SPOON SAMPLE	WS	WASH SAMPLE
TW	THIN WALL SHELBY TUBE SAMPLE	AS	AUGER (GRAB) SAMPLE
PH	SAMPLER ADVANCED BY HYDRAULIC PRESSURE	TP	THIN WALL PISTON SAMPLE
WH	SAMPLER ADVANCED BY SELF STATIC WEIGHT	PM	SAMPLER ADVANCED BY MANUAL PRESSURE
SC	SOIL CORE	RC	ROCK CORE
▼	WATER LEVEL	$SENSITIVITY = \frac{UNDISTURBED\ SHEAR\ STRENGTH}{REMOLDED\ SHEAR\ STRENGTH}$	

*HIERARCHY OF SOIL STRENGTH PREDICTION: **1)** LABORATORY TRIAXIAL TESTING. **2)** FIELD INSITU VANE TESTING. **3)** LABORATORY VANE TESTING. **4)** SPT VALUES. **5)** POCKET PENETROMETER.

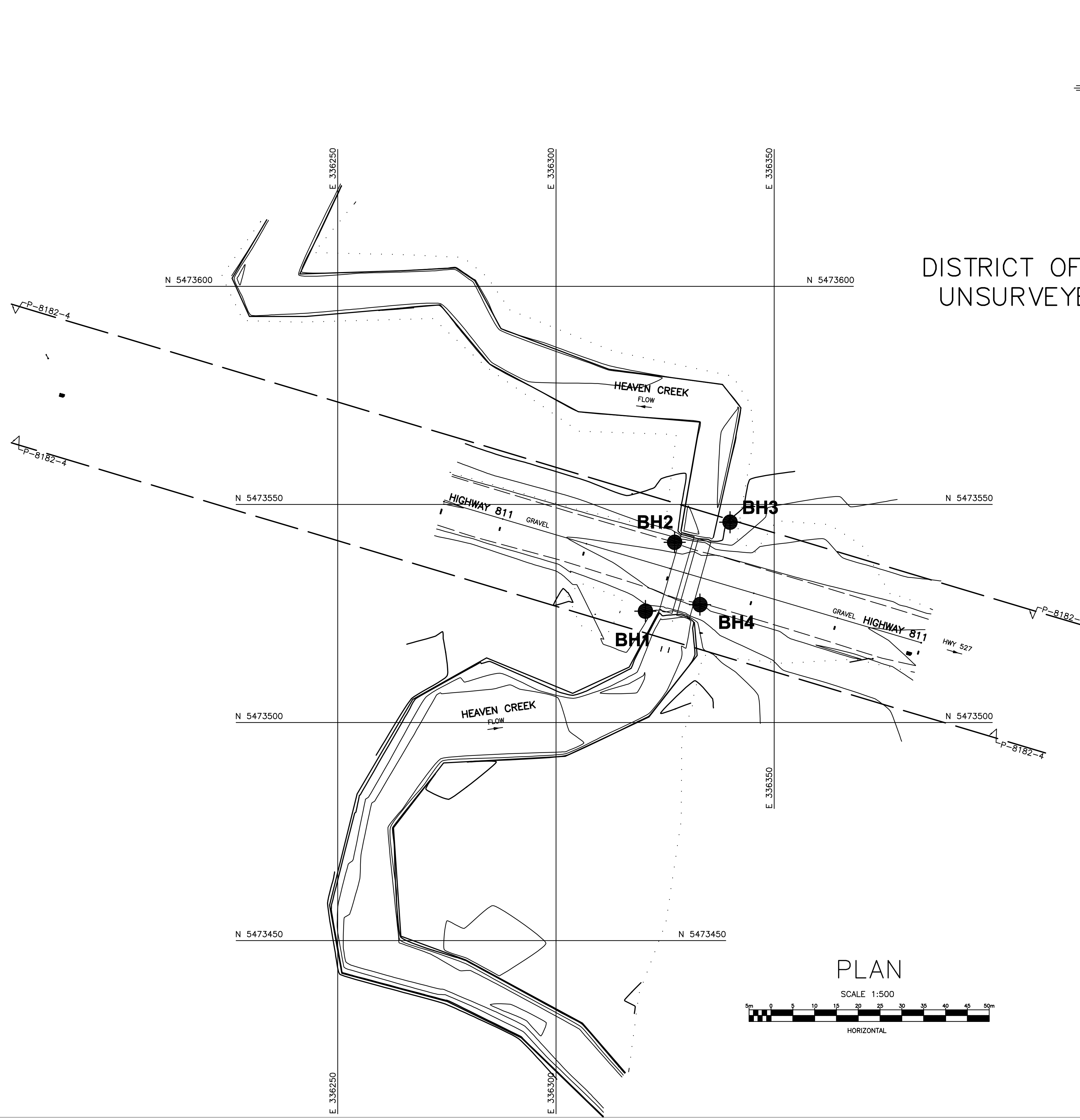
Appendix C

DRAWINGS

METRIC

DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLES OTHERWISE SHOWN. STATIONS IN KILOMETERS + METERS

DISTRICT OF THUNDER BAY
UNSURVEYED TERRITORY



PLAN

SCALE 1:500

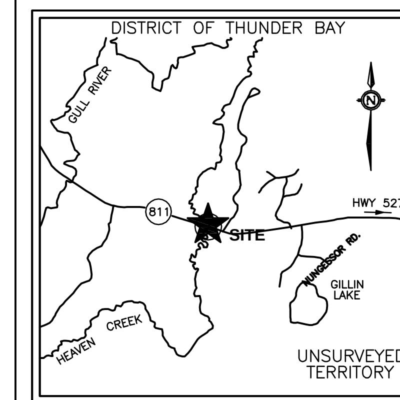
HORIZONTAL

CONT No 4014-E-0023
WP No xxx-xx-xx
SITE No 48W-197/C

CULVERT REPLACEMENT
HEAVEN CREEK

SHEET

Survey 13-06 Revised



PLAN

1.0km 0 1.0km

SCALE 1:50,000

KEY PLAN

LEGEND

Borehole

No.	Elevation	Northing	Easting	Station	Offset
BH1	448.0	5473525.5 m N	336320.5 m E	10+063	11.0 m LT
BH2	449.0	5473541.3 m N	336327.2 m E	10+061	6.0 m RT
BH3	447.9	5473545.9 m N	336339.9 m E	10+050	14.0 m RT
BH4	449.2	5473527.0 m N	336333.0 m E	10+051	6.0 m LT

NOTE:
The boundaries between soil strata have been established only at borehole locations. Between boreholes the boundaries are assumed by interpolation and may not represent actual conditions.

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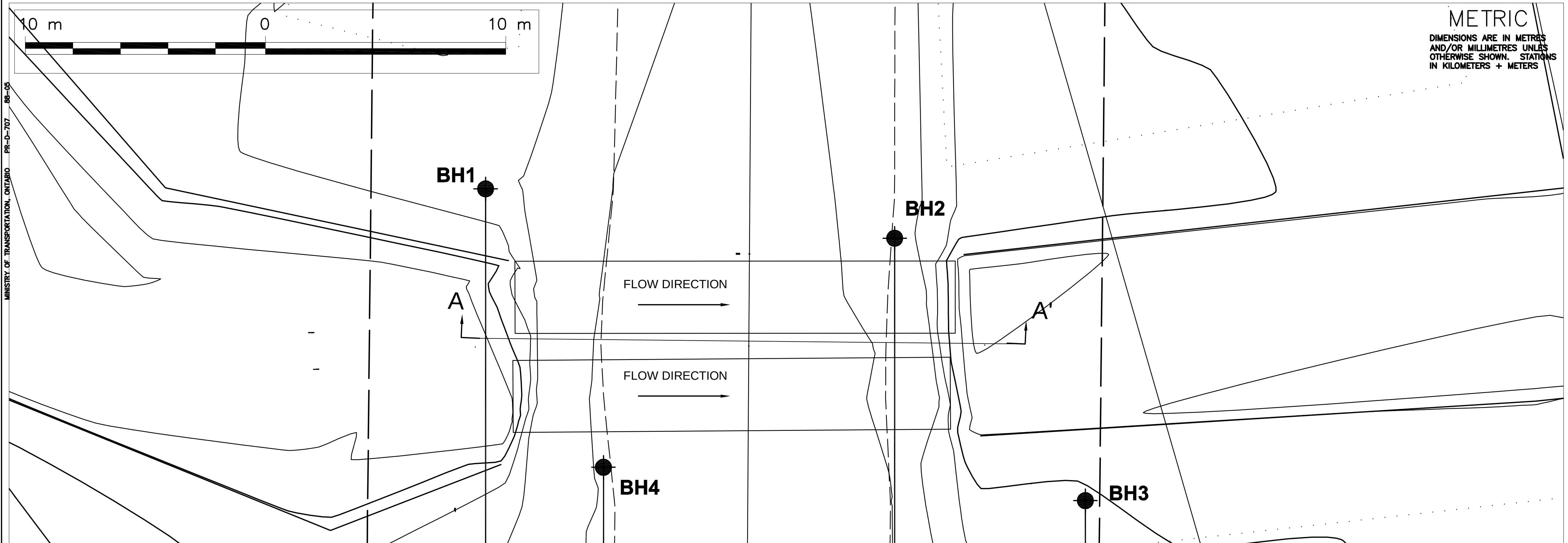
DRAWING 1

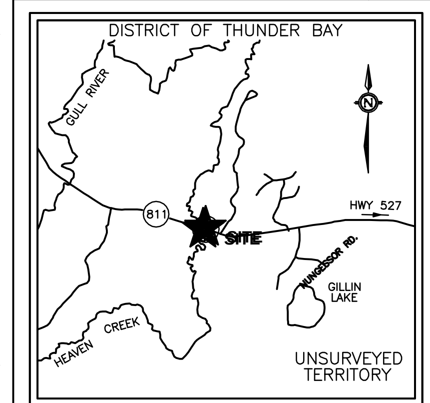
CONT No 4014-E-0023
WP No xxx-xx-xx
SITE No 48W-197/C

CULVERT REPLACEMENT
HEAVEN CREEK

SHEET

Survey 13-06 Revised

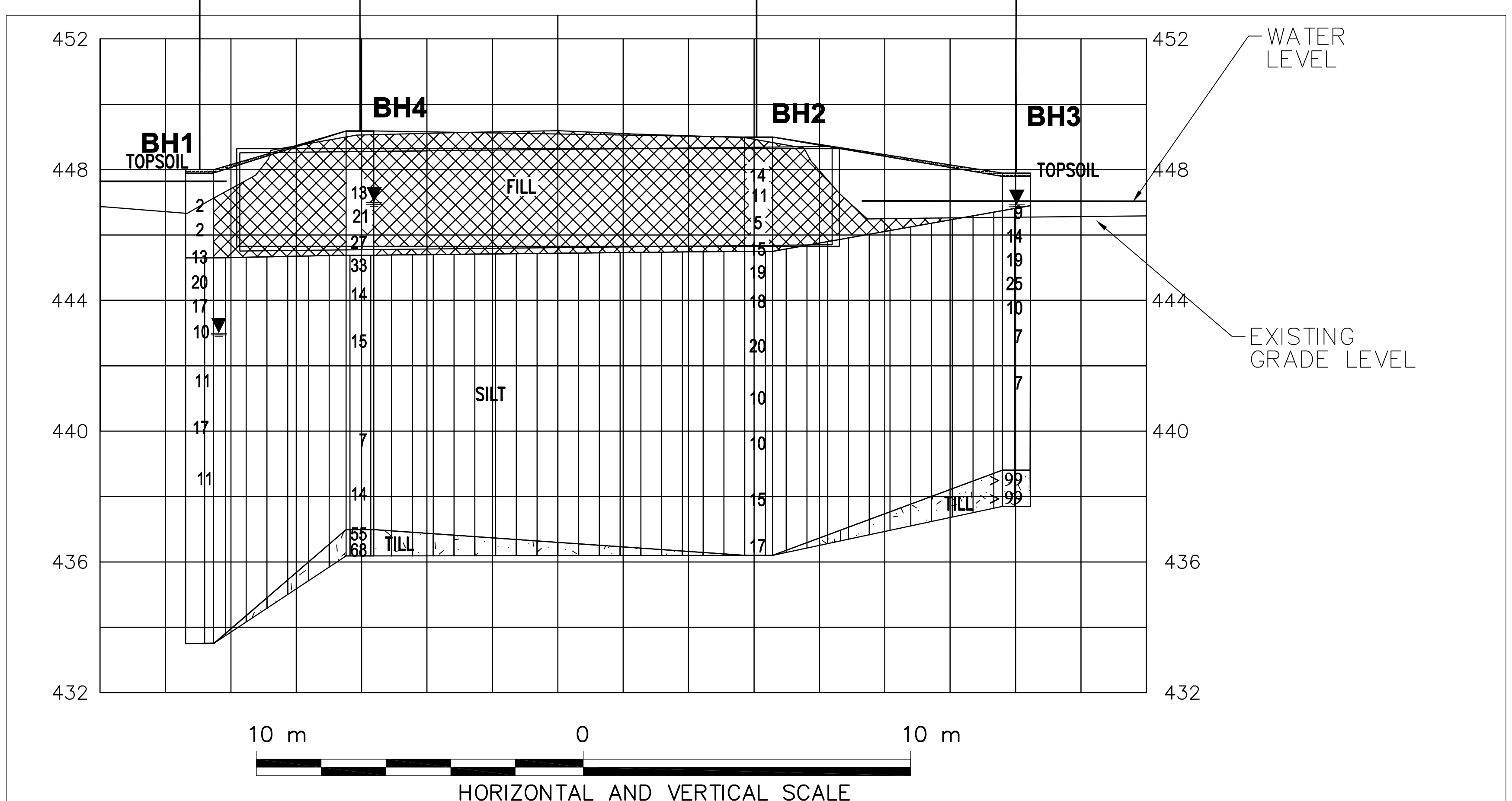




PLAN

1.0km 0 1.0km
SCALE 1:50,000

KEY PLAN



LEGEND

- Borehole
- 'N' Blows/0.3m (Std. Pen Test, 475 J/Blow)
- Groundwater Elevation

Fill, Organics, Topsoil, Till, Bedrock, Sand, Silt, Clay, Sand & Gravel, Boulders

No.	Elevation	Northing	Easting	Station	Offset
BH1	448.0	5473525.5 m N	336320.5 m E	10+063	11.0 m LT
BH2	449.0	5473541.3 m N	336327.2 m E	10+061	6.0 m RT
BH3	447.9	5473545.9 m N	336339.9 m E	10+050	14.0 m RT
BH4	449.2	5473527.0 m N	336333.0 m E	10+051	6.0 m RT

NOTE:
The boundaries between soil strata have been established only at borehole locations. Between boreholes the boundaries are assumed by interpolation and may not represent actual conditions.

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DRAWING 2

Appendix D

ENCLOSURES

Ministry of Transportation Ontario

RECORD OF BOREHOLE No BH1

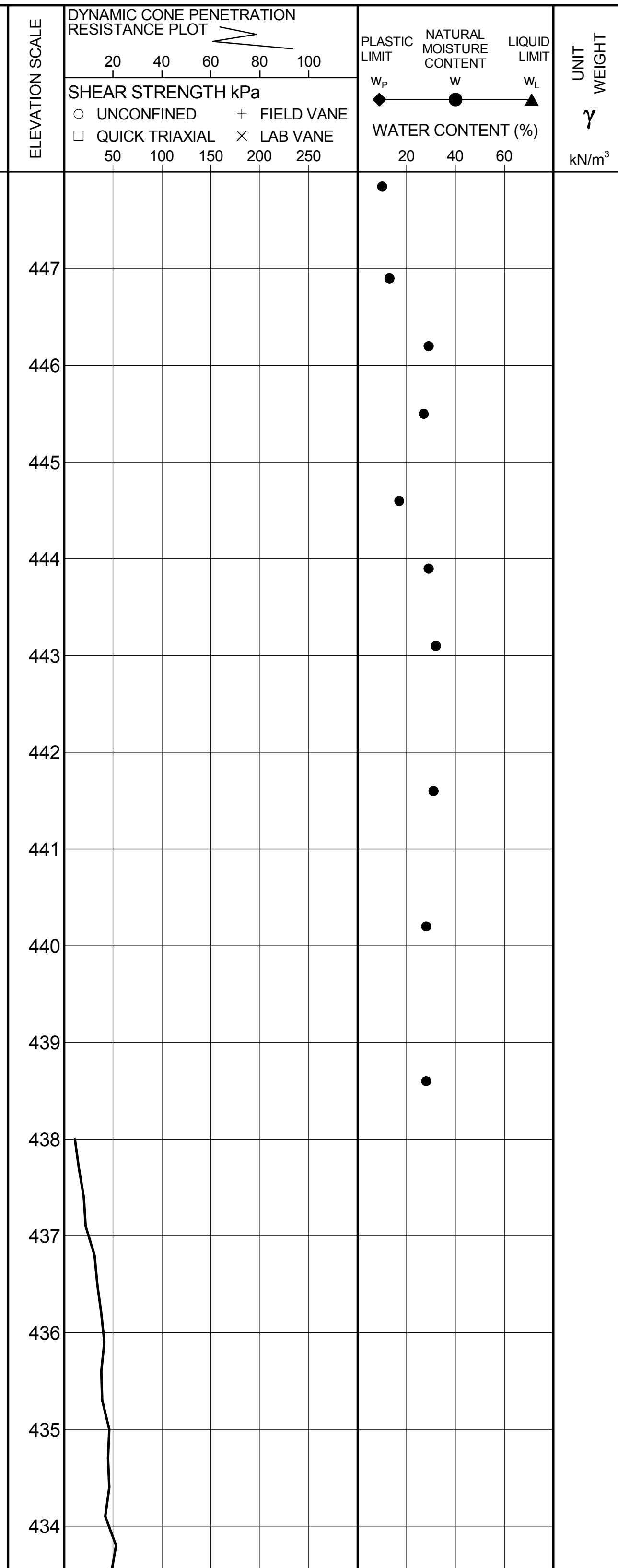
METRIC

W.P. GS-TB-020407 LOCATION HEAVEN CREEK CULVERT ORIGINATED BY PR

DIST MTO HWY 811 BOREHOLE TYPE HOLLOW STEM AUGER - 80 mm ID COMPILED BY SA

DATUM DATE 2015 09 19 CHECKED BY BV

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
448.0	GROUND SURFACE					
447.9 0.1	TOPSOIL	AS1	AS			
	FILL - SAND, SOME SILT, TRACE GRAVEL VERY LOOSE BROWN	SS2	SS	2		
	SAND, SOME ORGANICS	SS3	SS	2		
		SS4	SS	13		4 82 (14)
445.3 2.7	SILT AND SAND, TRACE GRAVEL COMPACT GREY	SS5	SS	20		3 46 (51)
		SS6	SS	17		
		SS7	SS	10	▽	
		SS8	SS	11		
		SS9	SS	17		
		SS10	SS	11		
433.5 14.5	END OF BOREHOLE AT 14.5 m					



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

Ministry of Transportation Ontario

RECORD OF BOREHOLE No BH2

METRIC

W.P.	GS-TB-020407	LOCATION	HEAVEN CREEK CULVERT	ORIGINATED BY	PR
DIST	MTO HWY 811	BOREHOLE TYPE	HOLLOW STEM AUGER - 80 mm ID	COMPILED BY	SA
DATUM		DATE	2015 09 18	CHECKED BY	BV

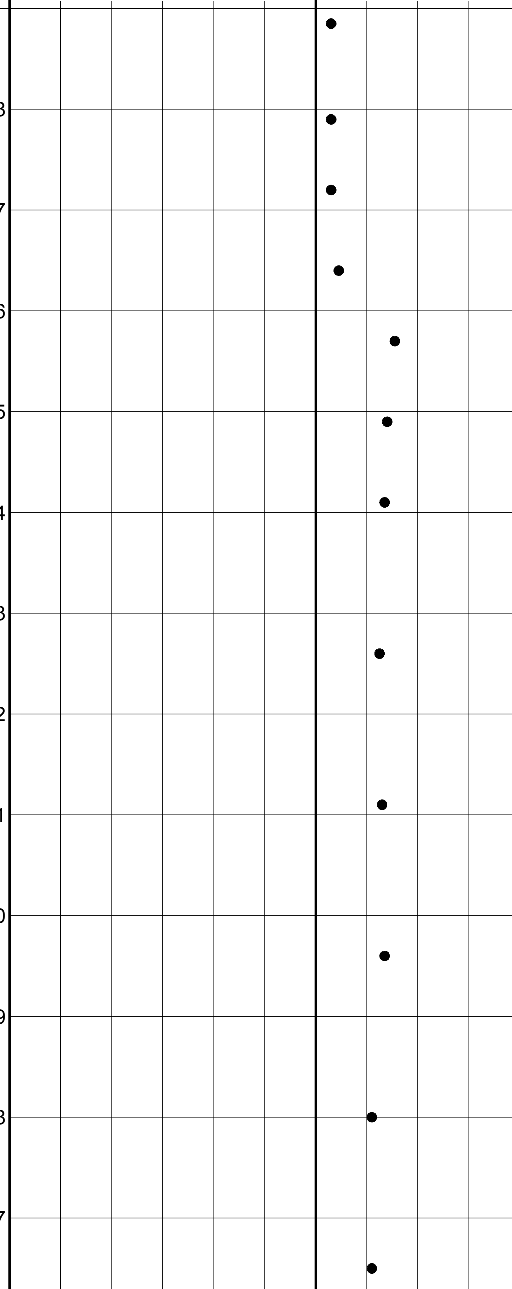
ELEV / DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
449.0	GROUND SURFACE						
	FILL - SAND, SOME GRAVEL, SOME SILT LOOSE TO COMPACT BROWN	AS1	AS				
		SS2	SS	14			
		SS3	SS	11			
		SS4	SS	5			
		SS5	SS	15			
445.5 / 3.5	SILT AND SAND COMPACT GREY						
		SS6	SS	19			
		SS7	SS	18			
		SS8	SS	20			
		SS9	SS	10			
		SS10	SS	10			
		SS11	SS	15			
		SS12	SS	17			
436.2 / 12.8	END OF BOREHOLE AT 12.8 m						

ELEVATION SCALE: 448, 447, 446, 445, 444, 443, 442, 441, 440, 439, 438, 437

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE □ QUICK TRIAXIAL × LAB VANE; 50 100 150 200 250

PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60



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

RECORD OF BOREHOLE No BH3

METRIC

W.P. GS-TB-020407 LOCATION HEAVEN CREEK CULVERT ORIGINATED BY PR

DIST MTO HWY 811 BOREHOLE TYPE HOLLOW STEM AUGER - 80 mm ID COMPILED BY SA

DATUM DATE 2015 09 19 CHECKED BY BV

ELEV DEPTH (m)	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
447.9	GROUND SURFACE						
447.8 / 0.1	TOPSOIL						
	FILL - SAND, SOME SILT, TRACE GRAVEL, LOOSE, BROWN	AS1	AS				2 79 (19)
446.9 / 1.0	SILT AND SAND, LOOSE TO COMPACT, GREY	SS2	SS	9	∇	447	
	SILT WITH ORGANICS	SS3	SS	14		446	
		SS4	SS	19		445	
		SS5	SS	25			
		SS6	SS	10		444	
		SS7	SS	7		443	
						442	
		SS8	SS	7		441	
		SS9	SS	13		440	
438.8 / 9.1	TILL - SAND, SILTY, SOME GRAVEL, VERY DENSE, GREY	SS10	SS	115+		439	19 42 (39)
		SS11	SS	134+		438	
437.7 / 10.2	END OF BOREHOLE AT 10.2 m						

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE □ QUICK TRIAXIAL × LAB VANE; 50 100 150 200 250

PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60

UNIT WEIGHT γ kN/m³

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

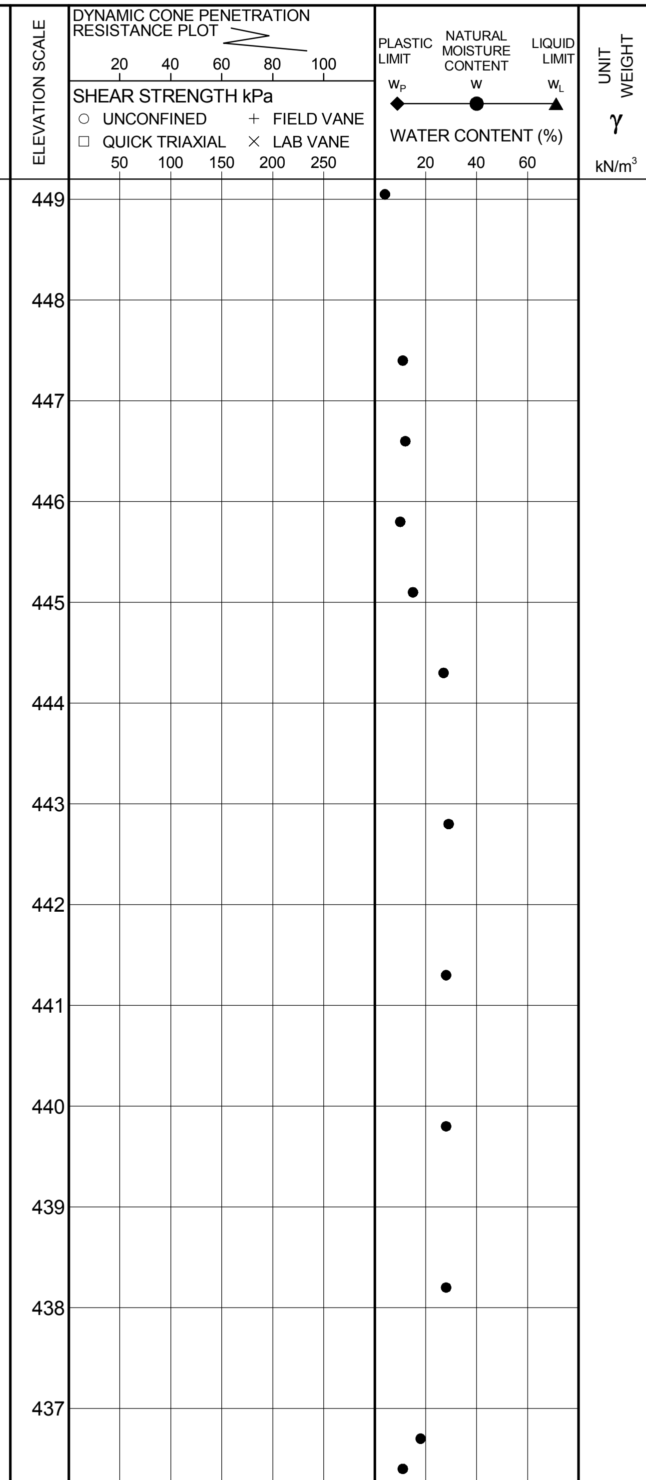
Ministry of Transportation Ontario

RECORD OF BOREHOLE No BH4

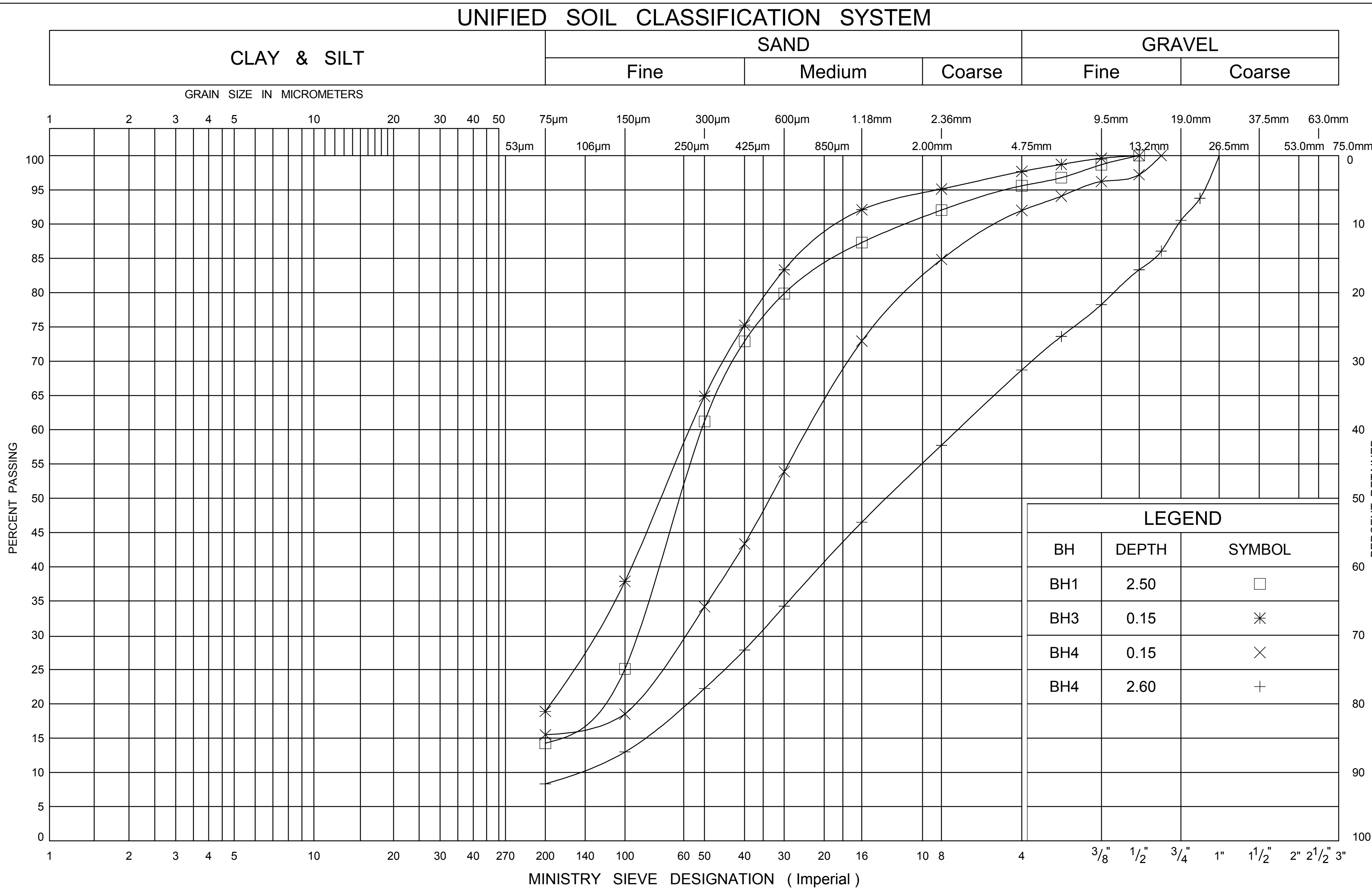
METRIC

W.P. GS-TB-020407	LOCATION HEAVEN CREEK CULVERT		ORIGINATED BY PR
DIST MTO HWY 811	BOREHOLE TYPE HOLLOW STEM AUGER - 80 mm ID		COMPILED BY SA
DATUM	DATE 2015 09 22		CHECKED BY BV

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
449.2	GROUND SURFACE					
	FILL - SAND, SOME SILT, TRACE GRAVEL COMPACT BROWN	AS1	AS			8 77 (15)
	ROCKFILL					
	SAND, GRAVELLY, TRACE SILT	SS2	SS	13	∇	
		SS3	SS	21		32 60 (8)
		SS4	SS	27		
445.4 / 3.8	SILT AND SAND LOOSE TO DENSE GREY	SS5	SS	33		
		SS6	SS	14		
		SS7	SS	15		
		SS8	SS	6		
		SS9	SS	7		
		SS10	SS	14		
437.0 / 12.2	TILL - SILT, SOME SAND, SOME GRAVEL VERY DENSE GREY	SS11	SS	55		
		SS12	SS	68		
436.2 / 13.0	END OF BOREHOLE AT 13.0 m					



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



LEGEND

BH	DEPTH	SYMBOL
BH1	2.50	□
BH3	0.15	✳
BH4	0.15	×
BH4	2.60	+

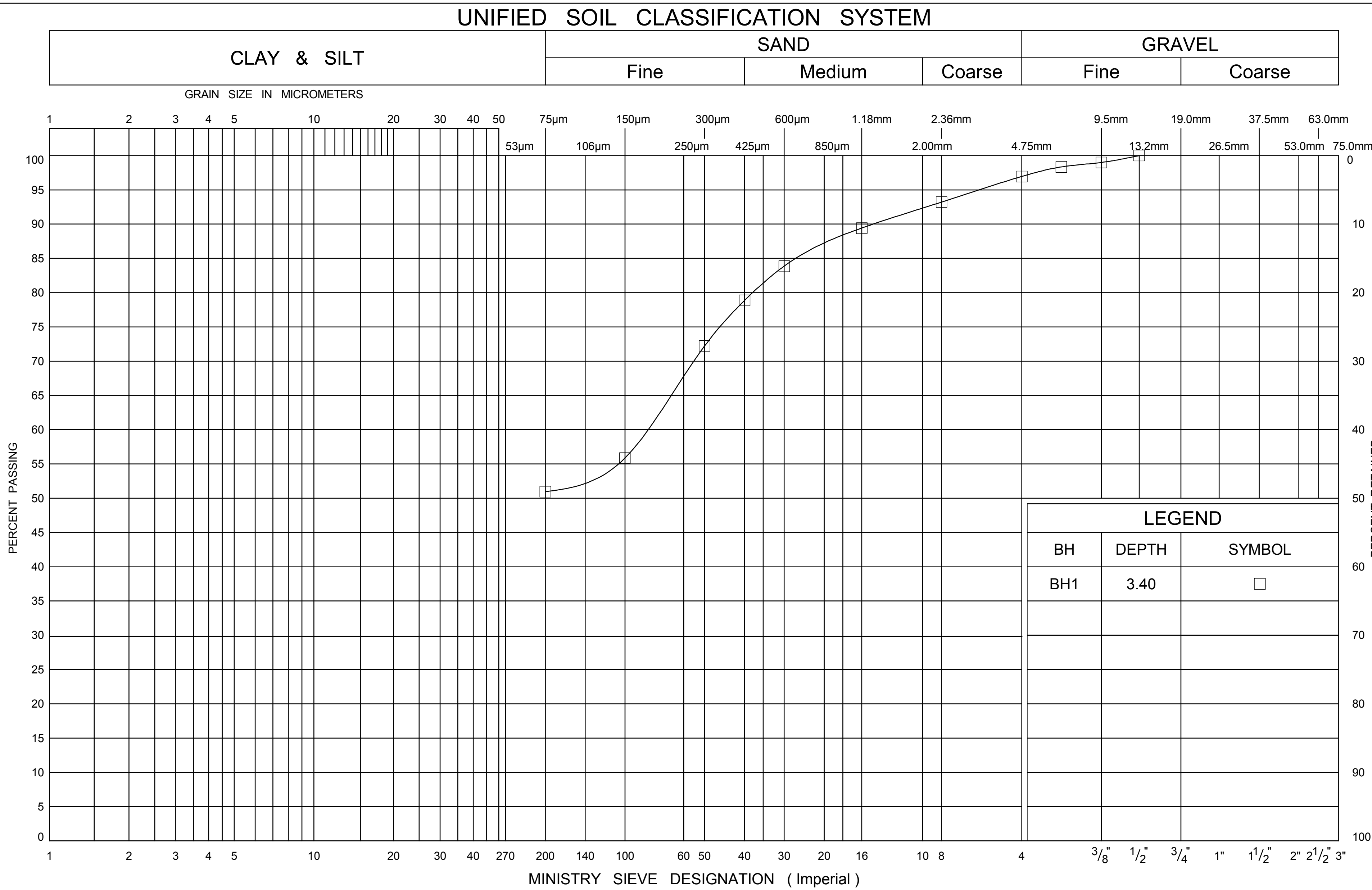
Ministry of Transportation Ontario

GRAIN SIZE DISTRIBUTION
SOIL DESCRIPTION
FILL

ENCLOSURE 5

DST REF. # GS-TB-020407

HEAVEN CREEK



Ministry of Transportation Ontario

GRAIN SIZE DISTRIBUTION

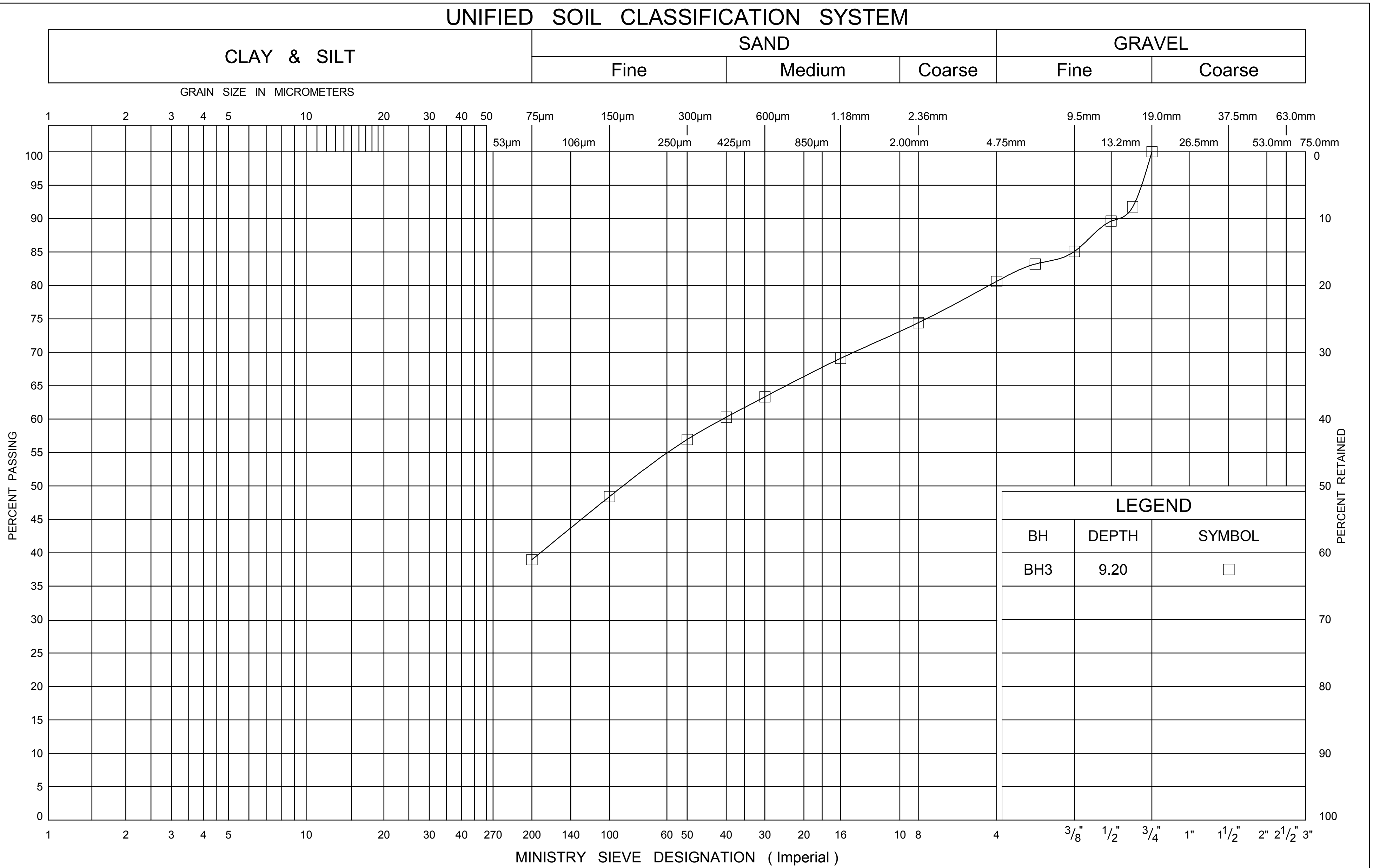
SOIL DESCRIPTION

SILT AND SAND

ENCLOSURE 6

DST REF. # GS-TB-020407

HEAVEN CREEK



Ministry of Transportation Ontario

GRAIN SIZE DISTRIBUTION
SOIL DESCRIPTION
TILL

ENCLOSURE 7

DST REF. # GS-TB-020407

HEAVEN CREEK