

Submitted To Triton Engineering Services Limited
18 Robb Blvd Unit 8,
Orangeville, Ontario L9W 3L2
On Behalf of the Ontario Ministry of Transportation

Culvert Replacement
Highway 7044
Site No. 46-390
GWP 5119-12-00

FINAL FOUNDATION INVESTIGATION AND DESIGN REPORT

Date: December 22, 2015
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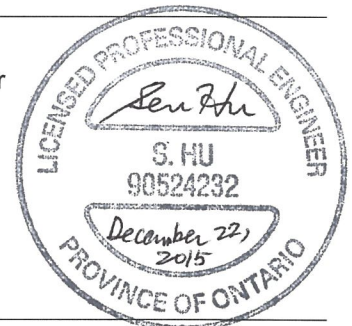
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Final Foundation Investigation and Design Report


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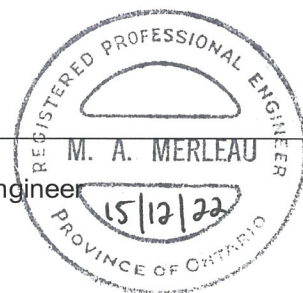


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1 INTRODUCTION

LVM-Merlex, a Division of EnGlobe Corp. (LVM-Merlex) has been retained by Triton Engineering Services Limited, on behalf of the Ministry of Transportation of Ontario (MTO), to carry out a foundation investigation at an existing centreline culvert site (Site No. 46-390) for the preliminary design of the required culvert replacement. The site is located on Highway 7044, some 6.7 km south of Highway 144, in the Township of Hart.

The foundation investigation location was specified by the MTO. The terms of reference for the scope of work are outlined in LVM-Merlex's Proposal for Foundation Engineering 11/11/11209, dated September 23, 2014. The purpose of this investigation was to determine the subsurface conditions in the area of the existing culvert. LVM-Merlex investigated the foundation area by the drilling of boreholes, carrying out in-situ tests, and performing laboratory testing on select samples.

2 SITE DESCRIPTION

Triple Corrugated Steel Pipe (CSP) culverts are located at the site of this foundation investigation, located on Highway 7044, some 6.7 km south of Highway 144, in the Township of Hart. For the purposes of this project, the intersection of the highway and culvert centerlines has been given a local site chainage of station 10+000, Township of Hart. The topography in the area of this site is generally rolling. The existing highway embankment currently supports two undivided lanes of highway, locally running in a north-south direction. The existing highway, at the culvert location, is constructed on a granular embankment some 2.1 m in height (at centreline), with centerline elevation of 98.2 m at the culvert location and has a granular surface. The existing embankment slopes, in the area of the culverts, have been generally established at slope angles of approximately 3H:1V. The culverts at this location consist of three 1220 mm diameter Corrugated Steel Pipe (CSP) culverts, some 12 m in length. Flow through the culvert is from the west to the east (right to left). Cobbles/boulders were observed in the stream bed, and in a berm along the south bank of the stream, to the west of the culvert (see Photo Essay, Appendix 4).

2.1 SITE PHYSIOGRAPHY AND SURFICIAL GEOLOGY

This project is located in the Geomorphic Sub-province known as the North Shore – Sudbury Ridges and Pockets. The topography on this section of Highway 7044 is generally rolling. Significant layers of earth overlay the bedrock. Organic materials were also observed. Within the project area native overburden consists primarily of sands overlying bedrock.

Bedrock in the area consists of massive granodiorite to granite.

3 INVESTIGATION PROCEDURES

The fieldwork for this investigation was carried out during the period of December 1st to 16th, 2014 during which time three (3) sampled boreholes and two testpits were advanced. Two (2) boreholes were advanced through the embankment at the location of the culverts, and a single borehole was advanced at the inlet (west) end of the culverts. One testpit was advanced at each of the inlet (west) and outlet (east) ends of the culverts.

The field investigation was carried out using a truck and bombardier mounted CME drilling rig equipped with hollow stem augers, standard augers, casing equipment and routine geotechnical sampling equipment. Testpits were advanced a CAT 360 tracked excavator operated by Belanger Construction Ltd. Soil samples were obtained at the borehole locations at regular intervals of depth using the standard 50 mm O.D. split spoon sampler advanced in accordance with the Standard Penetration Test (SPT) procedures (ASTM D-1586). The SPT method involves advancing a 50 mm O.D. split spoon sampler with the force of a 63.5 kg hammer freely dropping 760 mm. The number of blows per 300 mm penetration was recorded as the “N” value. When cohesive deposits were encountered, the in-situ strength was measured using an “N” size field vane, vane collar, and calibrated torque meter. All samples taken during this investigation were stored in labeled airtight containers for transport to our North Bay laboratory for visual examination and select laboratory testing.

Groundwater conditions in the open boreholes and test pits were observed during the advancement of and immediately following, completion of the individual boreholes. All open boreholes were backfilled upon completion with compacted auger cuttings in the general order they were removed, and where necessary, bentonite pellet backfill was added to the boreholes to bring them up to grade in accordance with requirements of Ontario Regulation 903. At the borehole(s) through the embankment, the upper portion of the hole, where necessary, was backfilled with an asphalt cold patch to seal the existing asphalt surface.

The fieldwork for this investigation was under the full time direction of a senior member of the LVM-Merlex engineering staff, who was responsible for locating the boreholes, clearing the borehole locations of underground services, in-situ sampling and testing operations, logging of the boreholes, labeling and preparation of samples for transport to our North Bay laboratory, plus overall drill supervision. All samples received a visual confirmatory inspection in our laboratory. Laboratory testing of select samples included routine testing for natural moisture content determination and particle size analysis. The results of the laboratory testing are presented on the individual Record of Borehole Sheets (Appendix 2), with a summary of results presented on the laboratory sheets in Appendix 3 (Figures Nos. L-1 to L-5 and Table No. L-6).

The location of the individual boreholes were determined in the field using temporary highway chainage (established by others) and offset relative to highway centerline. The centreline of this culvert was assigned Station 10+000. The MTO co-ordinates, northing and easting, were then

established for the boring locations. Elevations contained in this report are referenced to a geodetic datum, established by others.

4 SUBSURFACE CONDITIONS

Details of the subsurface conditions revealed by the investigation program are presented on the enclosed Records of Borehole Logs and Records of Testpit (Appendix 2) and on Drawing No. 2 (Appendix 3). Please note that stratigraphic delineation presented on the borehole logs and soil strata plot are the results of non-continuous sampling, response to drilling progress, the results of SPT, plus field observations. Typically such boundaries represent transitions from one zone to another and are not an exact demarcation of specific geological unit. Additional consideration should be given to the fact that subsurface conditions may vary markedly between adjacent boreholes and beyond any specific boring location, and are shown on the drawings for illustration purposes only.

4.1 CULVERT STATION 10+000, TWP OF HART

A plan and profile illustrating the borehole locations and stratigraphic sequences is shown on Drawing No. 2, Appendix 3. During the course of the exploration program, three (3) sampled boreholes and two testpits were put down at this site. Borehole Nos. 1 and 2 were advanced through the embankment, adjacent to the culverts. Borehole No. 3 and Testpit No. 1 were advanced adjacent to the culvert inlet, and Testpit No. 2 was adjacent to the culvert outlet. At the time of the subsurface investigation, the ground surface elevations at Boreholes Nos. 1 to 3 and Testpit Nos. 1 and 2 were recorded at elevations 98.1, 98.0, 96.7, 97.3, and 97.4 m, respectively.

4.1.1 Embankment Fill

At surface at Borehole Nos. 1 and 2, a layer of fill consisting of brown gravelly sand to sand and gravel to gravelly sand, trace silt, was penetrated. Cobble size rock pieces were encountered in the fill layer. The natural moisture content measured on samples of this deposit was in the order of 2 to 13%. Gradation analyses were carried out on two (2) samples of this deposit, the results of which indicated 30 to 41% gravel size particles, 50 to 61% sand size particles, and 9% silt and clay size particles (Figure No. L-1, Appendix 3). Based on SPT 'N' values of 15 to 35 blows per 300 mm penetration, the compactness of this deposit was described as compact to dense. This deposit was encountered to depths of 2.1 and 2.2 m below grade at Borehole Nos. 1 and 2, respectively (elevations 96.0 and 95.8 m, respectively).

4.1.2 Organic Soils

At surface at Borehole No. 3, a layer of organic soils, trace sand, trace silt, trace roots was penetrated. The natural moisture content measured on samples of this layer was in the order of 101%. This organic soil layer was encountered to a depth of 0.7 m below ground surface (elevation 96.0 m).

4.1.3 Sands and Gravels

Underlying the embankment fill at Borehole Nos. 1 and 2, underlying the organic soils at Borehole No. 3, and at surface at Testpit Nos. 1 and 2, a deposit of brown sandy gravel to gravelly sand, trace to some silt was penetrated. Cobble and boulder size rock pieces were encountered in this deposit. At Testpit Nos. 1 and 2, the upper 1.2 and 1.0 m of this deposit, respectively, contained organics and roots/rootlets. The natural moisture content measured on samples of this deposit was in the order of 13 to 22%. Gradation analyses were carried out on four (4) samples of this deposit, the results of which indicated 37 to 56% gravel size particles, 38 to 59% sand size particles, and 4 to 20% silt and clay size particles (Figure No. L-2, Appendix 3). Based on SPT 'N' values of 15 to 47 blows per 300 mm penetration, this deposit was described as compact to dense. This deposit was encountered to depths of 7.0, 7.2, and 5.6 m below grade at Borehole Nos. 1 to 3, respectively (elevations 91.1, 90.8, and 91.1 m, respectively). Sampling was encountered in this deposit at depths of 4.9 and 4.5 m below grade at Testpit Nos. 1 and 2, respectively (elevations 92.4 and 92.9 m, respectively).

4.1.4 Sands

Underlying the sands and gravels encountered at Borehole Nos. 1 to 3, a deposit of brown sand trace gravel trace silt was penetrated. The natural moisture content measured on samples of this deposit was in the order of 18 to 23%. A gradation analysis was carried out on one (1) sample of this deposit, the results of which indicated 2% gravel size particles, 96% sand size particles, and 2% silt and clay size particles (Figure No. L-3, Appendix 3). Based on SPT 'N' values of 7 to 29 blows per 300 mm penetration, this deposit was described as loose to compact. This deposit was encountered to depths of 9.9, 10.2, and 8.7 m below grade at Borehole Nos. 1 to 3, respectively (elevations 88.2, 87.8, and 88.0 m, respectively).

4.1.5 Sands and Silts

Underlying the sands at Borehole No. 3, a deposit of brown sand and silt trace clay was penetrated. The natural moisture content measured on samples of this deposit was in the order of 17 to 27%. A gradation analysis was carried out on one (1) sample of this deposit, the results of which indicated 0% gravel size particles, 52% sand size particles, 47% silt size particles, and 1% clay size particles (Figure No. L-4, Appendix 3). Based on SPT 'N' values of 0 (static weight of hammer) to 22 blows per 300 mm penetration, this deposit was described as very loose to compact. This deposit was encountered to a depth of 11.7 m below grade at Borehole No. 3 (elevation 85.0 m).

4.1.6 Gravelly Sands

Underlying the sand at Borehole Nos. 1 and 2, and underlying the sands and silts at Borehole No. 3, a deposit of brown gravelly sand some silt was penetrated. Cobble and boulder size rock pieces were encountered in this deposit. The natural moisture content measured on samples of this deposit was in the order of 8 to 12%. A gradation analysis was carried out on one (1) sample of this deposit, the results of which indicated 33% gravel size particles, 55% sand size particles, and 12% silt and clay size particles (Figure No. L-5, Appendix 3). Based on SPT 'N'

value of 37 blows per 300 mm penetration, this deposit was described as dense. Sampling was terminated in this deposit at depths 10.7, 12.2, and 12.4 m below grade at Borehole Nos. 1 to 3, respectively (elevations 87.4, 85.8, and 84.3 m, respectively).

4.2 GROUNDWATER DATA

At the time of this investigation (December 4, 2014), flow was observed through the culvert from west to east. The water level at the culvert inlet was measured at elevation 96.6 m.

Measurements of the groundwater table and cave-in levels were undertaken, where possible, in the open boreholes during the advance of the individual borings and upon completion. These levels are recorded on the individual Record of Borehole Log Sheets (Appendix B).

The water levels were measured at elevations 96.9 (December 3, 2014), 96.7 (December 11, 2014), and 96.3 m (December 16, 2014), at Borehole Nos. 1, 2 and 3, respectively.

The groundwater and river water levels will fluctuate seasonally/yearly.

5 DISCUSSION AND RECOMMENDATIONS

5.1 GENERAL

A foundation investigation was carried for the proposed replacement of triple CSP culverts as identified by the MTO.

The existing culverts, located at Station 10+000, in the Township of Hart, are 1220 mm diameter CSP culverts some 12 m long. The existing culvert invert at centerline is at a depth of some 2.1 m (elevation 96.1 m). The existing highway embankment currently supports two undivided lanes of highway, running in a north-south direction. The highway has a gravel surface. Flow through the culvert is from the right to the left (west to east). Based on data from this foundation investigation, the embankment supporting the existing pavement structure at this site has been constructed using a crushed stone surface over gravelly sand to sand and gravel fills. The native material, underlying the embankment fill, generally consisted of compact to dense sandy gravels to gravelly sands overlying loose to compact sands, underlain by dense gravelly sands. Cobbles and boulder were encountered in the fills and native deposits. Advancing hollow stem augers through these deposits was difficult and NQ size casing was generally required to advance through the cobbles and boulder encountered in the deposits. Coring was required at Borehole Nos. 1 and 2.

It is understood that preliminary plans call for a single cell concrete box culvert (closed or open type), with a width in the order of 6 to 8 m, is being considered to replace the existing CSP culverts. It is understood that the new culvert will be constructed along a similar skew and alignment. It is further understood that the vertical alignment of the highway will be raised by some 1 m. As a result, the embankment and culvert must also be widened/lengthened.

5.2 EMBANKMENT WIDENING CONSIDERATIONS

The native soils in the area of the culvert generally consist of compact to dense gravelly sands to sands and gravels. A layer of organic soils, some 0.7 m thick, was encountered at the ground surface at location of Borehole No. 3.

The grade raise and widening can be constructed to match the existing embankment in the culvert area. Where organics soils are encountered, they must be removed to native mineral soils prior to construction of the widening and placing the culvert.

Embankment side slopes in granular fill should be established at an angle of 3H:1V, as per OPSD 200.010 for new construction, however since this work is limited to the area of culvert the slopes could be reconstructed to match existing at 2H:1V, up to the underside of pavement structure. Pavement design recommendations are provide in the separate Pavement Design Report, LVM-Merlex Ref No. 11/11/11209-P3, dated March 4, 2015. Embankment fill within the depth of frost penetration of 2.1 m must be OPSS 1010 Select Subgrade Material (SSM) quality or better.

5.3 FOUNDATION CONSIDERATIONS

The founding native dense sands present below the existing embankment are considered adequate for support of a culvert and for a conventional highway embankment of this height. Bearing resistance should not be a major issue provided the natural bearing surface is not disturbed during construction and groundwater is controlled throughout construction, as discussed in Section 5.7.

Three types of culvert replacement (i.e. Steel Sheet Pile Wall Culvert, Open Footing Culvert and Twin Precast Concrete Box Culvert) are proposed by the structural engineer. Based on the preliminary structure design drawings, the span width of the new culvert will be some 8 m for the proposed Steel Sheet Pile Wall Culvert and Twin Precast Concrete Box Culvert. The span width of the new culvert will be some 8.9 m for the proposed Open Footing Culvert. The width of the strip footing will be some 2.4 m if the open footing culvert is considered.

Based on the characteristics of the native sand subgrade present below the culverts, the response of the existing embankment, and a founding elevation similar to that of the existing culverts, a factored bearing resistance at ULS of 650 kPa can be used for a closed culvert (i.e. precast concrete frame box culvert). In consideration of the width of the culvert, depth of overburden (assumed culvert invert at elevation 95.5 m), and response of the existing embankment, a geotechnical reaction at SLS of 300 kPa can be used for design, in consideration of 25 mm settlement.

If open culverts (i.e. concrete frame open culverts, with wall footings) are considered, then a factored bearing resistance at ULS of 200 kPa, and a geotechnical reaction at SLS of 150 kPa would apply for design of strip footings at depth (assumed footing elevation at 94.3 m), in consideration of 25 mm settlement and taking into consideration the limited depth of overburden and smaller footing width.

Consideration of selecting types of concrete frame culverts is discussed in Section 5.4.1.

5.3.1 Sheet Pile Wall Culvert

From an environmental viewpoint, it is understood that a Steel Sheet Pile Wall Culvert is under preliminarily considered, to avoid new foundation excavation and associated dewatering system. Based on the preliminary structural loading requirement of 390 kN/m at ULS along the sheet pile wall, the sheet pile wall shall penetrate at least 3 m (not greater than Elevation 85 m) or refusal in the lower gravelly sand deposit. Drivability and the local construction experiences of driving sheet piles using a vibratory hammer have been discussed among the project team and with of a local contractor. Sheet piles, with a sufficiently robust cross section, have been driven into similar gravelly sand to sand and gravel deposits using a relatively large vibratory hammer. The empirical Hiley formula, traditionally used for conventional drop and diesel hammers, is not appropriate for estimating the geotechnical resistance during vibratory driving of sheet piles. Vibrators are typically hydraulic with counter-rotating eccentric masses located within the driving assembly or exciter block. A few empirical formula have been suggested to

estimate the geotechnical resistance of piles driven by the vibratory pile driver. However, there is limited published testing data for the vertical capacity of sheet piles and the available project data where sheets have been used to offer vertical resistance, the sheets were driven to refusal on bedrock.

The wave-equation theory is still applicable as the fundamental principle of the mechanism for sheet pile driving; however the induced local soil liquefaction during pile driving and the frictions between the interlocks of sheet piles make it very difficult to estimate the geotechnical resistance of the driven sheet piles. It would be necessary to carry out in-situ testing and measurement (e.x. Vibratory Driving Analysis (VDA)) to correlate the setting criteria and the geotechnical resistance of the sheet piles verified by the static pile load test or PDA testing. A trial driving test of sheet piles undertaken to demonstrate its drivability, check the refusal depth and required setting criteria for driving the sheet piles is recommended to be completed during Detailed Design of the project as described below.

- Drive the test sheets with a conventional crane equipped with leads and boom for a diesel or drop hammer. The piles could be advanced with a vibratory hammer, however testing has to be carried out with a drop or diesel hammer;
- Allow the piles to set over-night. The pore water pressure should dissipate after 12 hours or more in the sand and gravel with cobble deposit encountered at site;
- Test and establish the vertical geotechnical resistance of the tested sheet piles with the diesel/drop hammer and Pile Dynamic Analysis (PDA) or static load test as per ASTM D1143.

A setting criteria (i.e. required energy rate of hammer and its advance rate) will be established for the sheet piles driven to the anticipated founding and/or refusal level after the trial testing.

5.3.2 Slope Stability

A stability analysis, using the GEO-SLOPE computer program, Slope/W (GeoStudio 2007, version 7.17, Geo-Slope International Ltd.), was carried out at this location with embankment slopes on an inclination angle of 3H:1V in embankment fill, and the raised grade. For the purposes of these analyses, the materials were modeled using the following geotechnical parameters:

PARAMETER	MATERIAL		
	EMBANKMENT FILL	SANDS AND GRAVELS/ GRAVELLY SANDS	SANDS
Unit Weight (kN/m ³)	20	19	18.5
Effective Friction Angle (degrees)	32	32	30

The unit weights and friction angles for the slope calculations are based on general representative values for the various soil types, obtained through laboratory testing and tactile analysis. The results of the analyses indicated a factor of safety for the new embankment in the order of 1.8 (see Figure No. S-1, Appendix 5). The results of the analyses indicated a factor of safety for the new embankment slope with an inclination angle of 2H:1V in the order of 1.4 (see Figure No. S-2, Appendix 5). The minimum allowable factor of safety for embankment slopes is generally taken at 1.3. Lower factors of safety will occur during excavation and backfilling as discussed in Section 5.5. Short term stability should not be an issue if construction is carried out as described herein. The long term stability of the new embankment will not be an issue provided it is properly reconstructed.

5.4 CULVERT DESIGN, BEDDING, AND EMBEDMENT

The embankment consists of granular fills. The results of this investigation indicate that, below the culvert invert and the organic soils, the native soils at Borehole Nos. 1 to 3 consisted of compact to dense sands and gravelly sands overlying loose to compact sands, underlain by dense gravelly sands. The existing embankment has preloaded the soils at the culvert locations, however, it is understood that the vertical alignment will be raised. The grade raise and widening of embankment will induce additional loads on the subsurface soils. Considering a grade raise of some 1 m, and generally dense sands and gravel subgrade, the resulting settlement of the embankment raise/widening will be negligible. Therefore, installing the culvert on a camber will not be required at this site.

5.4.1 Rigid Concrete Culvert

It is understood that a precast concrete rigid frame box culvert is the preferred option for culvert replacement at this site. Bedding for the closed type of a rigid frame box culvert shall consist of Granular A with a thickness of 300 mm. The bedding under the middle third of the box unit base should be loosely placed and uncompacted. The upper 75 mm portion of the Granular A bedding should be uncompacted throughout the length/width of the box and incorporated as the top levelling course in conformance with OPSS 422. Alternatively, specifically if construction is carried out under wet conditions, a 19 mm clear stone bedding and levelling coarse should be used, which would aid in dewatering applications. During backfilling the embankment fill should be placed in a balanced manner on the outer sides of the box unit. The elevation difference of the backfill on either side of the box unit must be limited to a maximum of 400 mm. Backfilling and construction of pre-cast concrete box culverts shall be in accordance with OPSS 422.

Cover material for concrete box culverts can consist of Granular A, placed to the dimensions as shown on MTOD-803.021.

The joints between precast box units should be covered, at a minimum, with a strip of Non-Woven Class II Geotextile 600 mm in width, centered over the joint, covering the top of the culvert and extending halfway down the sides of the culvert to prevent the infiltration of fines.

Apron (cut-off) walls, 1.2 m deep, must be added to the ends of the rigid frame box culvert in accordance with the MTO Concrete Culvert Design Manual.

The inlet and outlet stream bed shall be protected with a full apron of Rip Rap (R-50 size as per OPSS 1004). Depending on the hydraulic review/flood velocities, a rounded Stream Bed (SB) material may be suitable to use at the culvert inlet and outlet stream bed. The apron shall extend 3 m beyond the sides of the culvert, be 500 mm thick and extend across the stream bed and up the embankment to 600 mm above the High Water Level. Clay seals are generally used where significant head differences exist between the inlet and outlet of a culvert to prevent flow through the embankment. Considering the anticipated slope of the culvert invert, at this location, clay seals are not considered necessary.

It is further understood that the open frame rigid concrete culvert(s) are also under consideration for culvert replacement at this site. Footings for open culverts must be protected against the detrimental effects of scour and frost heave. As per the CHBDC the minimum depth of footing for scour protection is not less than 1.5 m, below the original stream bed. The MTO Concrete Culvert Design and Detailing Manual, August 2003, Division 5 RFO Culverts, indicates that the standard footing depth is 1.2 m or equal to the minimum depth of frost protection in Ontario. This implies the depth of footing is to be increased to match the depth of frost penetration at the specific site. The frost penetration below pavement is estimated at 2.1 m below unheated structures and cleared surfaces.

Excavations to these depths, or greater, will require a sophisticated groundwater control system to maintain the native cohesionless soils, (sand and gravel a high concentration of cobbles and boulders), in an unwatered stable condition during construction. Penetration of a sheet pile system, for groundwater control, may be difficult due to the high concentration of cobbles and boulders in the underlying native soils and the dense state of compactness. Installation of well points or larger diameter wells may also encounter difficulties, due to the cobbles and boulders, and would probably require, at a minimum, a duplex rotary percussion drilling system to install the filtered well points with filter packs to the proper depth.

Besides foundation depth, scour protection can also be supplied through the placement of R-50 Rip Rap, 500 mm deep to line the stream bed, in the bottom of the open culvert, with an apron of Rip Rap, extending 5 m beyond the inlet and outlets, 5 m either side out the culvert ends and up the bank to 600 mm above the high water level. Depending on the hydraulic review, a Stream Bed (SB) material may be suitable to use at the culvert inlet and outlet stream bed. This protection would allow the footings to be founded at a higher elevation. However, raising the

footings will place them within the potential depth of frost penetration and alternative means of frost protection (i.e. non-frost susceptible fill, synthetic insulation, etc.) may have to be considered, or the culvert designed to withstand the possible movements due to differential frost heaving.

Alternatively closed sheet piles may be used to prevent scour of the strip footing. The strip footing could be enclosed with sheet piling, driven to sufficient depth to control groundwater/boiling condition during excavation. The sheet piles of stream side and part of the ends could be left in place to prevent scour of the footing. The footing elevation could also be raised, in combination with synthetic insulation for frost protection. Considering the potential difficulties (depth of excavation and groundwater control) with the construction of footings for an open rigid frame concrete culvert, a precast rigid frame concrete box culvert is considered to be more appropriate at this site.

5.5 CULVERT INSTALLATION AND CONSTRUCTION STAGING CONSIDERATIONS

The invert elevation of the existing culvert is at 96.1 m, with the top of the embankment at elevation 98.2 m at centerline. As such, the embankment at this location is some 2.1 m in height above the culvert invert at the centerline. Therefore, a minimum 3.0 m deep excavation (i.e. to elevation 95.2 m) will be required, at centreline, in consideration of countersinking the box culvert some 300 mm, a 300 mm thick concrete culvert bottom, and a 300 mm thick layer of bedding material. It is understood that the preferred construction method of carrying out the culvert replacement is to temporarily close the roadway to allow culvert replacement with open cut excavation, if applicable. However, staged excavation and roadway protection will be discussed.

The present platform width at this location is some 9.5 m as can be seen on the cross section on Drawing No. 2. The platform width at this location, as is, will not be sufficient to carry out an open excavation using staged construction unless local lowering of the grade and/or embankment widening is undertaken. If sufficient lowering or widening cannot be accommodated then consideration can be given to constructing a vertical wall for use as a protection system.

5.5.1 Staged Construction

As noted, the platform at this location, as is, is of insufficient width to carry out an open excavation using staged construction if required, unless temporarily lowering of the vertical alignment or embankment widening is carried out. To carry out an open cut excavation, locally lowering the grade to allow for staged construction using staged sequencing and limiting traffic flow to one lane would be required.

A possible staging plan for a continuous open cut excavation under a 24/7 traffic control operation, is as follows:

- Locally lower the grade or widen the embankment at the culvert location, as required.

- Limit traffic to a single lane on the left, with a minimum platform width of 6 m, under 24/7 traffic control.
- Open cut excavate, to the right.
- Reconstruct the embankment on the right, with a minimum platform width of 6 m for traffic.
- Divert the single lane of traffic to the right and continue open excavation to install the remainder of the culvert on the left.
- As the width of the platform increases on the right, the vertical alignment can be raised as per design, and the traffic can revert back to two lanes when sufficient width permits.

5.5.2 Protection System

As noted above, consideration should be given to constructing a vertical wall if required, along centerline, for use as a temporary protection system.

The installation of a protection system, if required, for use in the culvert replacement operation will require penetration through some 2.1 m of fills. The embankment fills are generally underlain by compact to dense sands and gravels. As noted, cobbles/boulders size rocks were encountered in the embankment fills and native sands and gravels. As discussed in Section 5.3, advancing a temporary retaining system in consideration of the anticipated soil conditions presenting cobble/boulder size rock pieces in the embankment fills and native soils may consider the use of sheet piles with sufficiently robust cross section for a protection system based on previous construction experiences in the region of the north Ontario. Several approaches to constructing a protection system are described in the following. See Table A, Appendix 5, for advantages and disadvantages for the different type of protection system considered for this site.

One alternative method to construct a protection system would be to penetrate the cobble fill in the embankment with H piles (soldier piles) extending into the underlying sands and gravels and install lagging with an appropriate dewatering system. Pre-drilling may likely be required to advance the H piles through the fill and native soils. The H piles would be installed at an interval of 2.5 to 3 m apart and the lagging would be installed as the excavation progresses. A waler and raker system, or tie back system may have to be installed as the excavation advances. The contractor must be prepared to address large pieces of rock and control groundwater as the excavation progresses, without compromising the adjacent active lane of traffic.

The resistance (R) for grouted anchors, located outside the active failure wedge, in cohesionless soils can be estimated from the following equation as supplied in the Canadian Foundation Manual (4th Edition):

$$R = \sigma'_z * A_s * L_s * \alpha_g \quad \text{Where:} \quad \sigma'_z = \text{effective vertical stress at the midpoint of the load carrying length}$$

A_s = effective unit surface area of the anchor

L_s = effective embedment length of the anchor

α_g = anchorage coefficient use 1.0 for granular backfill

Unless the pull-out resistance (capacity) of the anchor is proven with a load test program, the allowable anchor load (as suggested by the Canadian Foundation Engineering Manual, 4th Edition), is commonly obtained by dividing the computed capacity of the anchor by a factor of safety of 3. Alternatively, proprietary anchor systems can be used.

Alternatively, a caisson wall or drilled micropile system with an intermediate support system of reinforced shotcrete, to act as lagging, could be considered for roadway protection at this site. However this shoring system is generally more costly.

The contractor's shoring/protection system design must be carried out by a geotechnical engineer with appropriate experience.

The protection system can be designed using the lateral earth pressure parameters as outlined in Section 5.6.

Considering the cohesionless nature of the embankment fills (granular pavement structure overlying a granular fill and rock fill mix) a rectangular apparent pressure distribution over the height of the cut would be appropriate for design of the temporary shoring. The width of the apparent rectangular pressure distribution, over the height of excavation, can be considered equal to $0.65 \cdot K_a \cdot \gamma \cdot H$, where:

K_a = active earth pressure,

γ = unit weight, and

H = height of wall above the base of excavation.

The temporary protection system should be designed and constructed to comply with OPSS 539. In consideration of the location of the protection system and traffic volume, a Performance Level 2 is considered appropriate.

5.6 LATERAL EARTH PRESSURES

Lateral earth pressures should be computed in accordance with the Canadian Highway Bridge Design Code (CHBDC). The design parameters for the bedding/embedment and backfill materials are as follows:

PARAMETER	GRANULAR A	GRANULAR B TYPE I	EMBANKMENT FILL	SANDS AND GRAVELS	SANDS
Unit Weight (kN/m^3)	22.8	21.2	20	19	18.5
Angle of Internal Friction	34°	31°	32°	32°	30°
Coefficient of Active Earth Pressure (K_a)	0.28	0.32	0.31	0.31	0.33

PARAMETER	GRANULAR A	GRANULAR B TYPE I	EMBANKMENT FILL	SANDS AND GRAVELS	SANDS
Coefficient of Passive Earth Pressure (K_p)	3.54	3.12	3.25	3.25	3.00
Coefficient of Earth Pressure at Rest (K_o)	0.44	0.48	0.47	0.47	0.50

For rigid structures, such as a precast concrete culvert, deflection cannot occur, as such the “at-rest” condition (K_o) applies.

5.7 EXCAVATION, DEWATERING, AND EMBANKMENT RECONSTRUCTION

All excavations greater than 1.2 m in depth must, at a minimum, be sloped or shored in accordance with the Occupational Health and Safety Act Regulations for Construction Projects. The embankment material, above the water table, is considered a Type 3 soil as defined in the Occupational Health and Safety Act and Regulations for Construction Projects. Temporary open excavations above the groundwater table, could be cut back at an angle of 1H:1V, provided they are monitored continuously, however, below the groundwater table, the side slopes will have to be cut back to an angle of 2H:1V, possibly shallower, dependent upon the Contractors’ chosen method of controlling the groundwater. Temporary open cuts with a slope of 1H:1V cannot be left unattended (i.e. overnight, during breakdowns, etc.). If work must stop for extended periods of time, the temporary slopes must be flattened to a minimum angle of 1.5H:1V (above water level).

The excavation backfill should consist of Select Subgrade Material (SSM), at a minimum, above the culvert cover material up to the underside of the pavement structure. An SSM material must be used within the depth of frost penetration. Final (permanent) embankment side slopes in granular fills should be established to match the existing slopes or as per OPSD 200.010. Final slopes should be treated with a mulch and seed to prevent ravelling.

Bedrock was not encountered at the borehole locations within the anticipated depth of excavation, therefore bedrock excavation and/or blasting operations are not anticipated. Excavations must be maintained in a dewatered condition during excavation and foundation construction, and every reasonable effort must be made to prevent disturbing (piping/boiling) at the founding subgrade. Groundwater control, in accordance with OPSS 517 and 518, will be required to maintain a stable subgrade during culvert installation.

The water level in the creek was recorded at elevation 96.6 m at the culvert invert at the time of this investigation and excavations to an approximate elevation 95.2 m (at centreline) will be required to install a box (closed) culvert and bedding. A deeper, and subsequently more costly excavation will be required if an open culvert is considered. As such dewatering will be required during excavation and culvert installation. Considering the groundwater levels and subsurface conditions at this site, dewatering will be critical at this site.

During construction, installation of filtered sumps and pumping from the base of the excavation will, at a minimum, be required to maintain the excavation in a dewatered condition during

subgrade preparation and culvert installation. However, this method of groundwater control is generally only effective when the groundwater in the excavation is less than a depth of some 1 m above the final base of the excavation. To effectively lower the groundwater to a greater depth, a more sophisticated groundwater control system, such as a well points or closed sheeting, would have to be considered.

At this site, excavations will likely require a sophisticated groundwater control system to maintain the native cohesionless soils, (sands and gravels a high concentration of cobbles and boulders), in an unwatered stable condition during construction. Adequate penetration of a sheet pile system, for groundwater control, is likely not feasible due to the high concentration of cobbles and boulders in the underlying native soils and the dense state of compactness. Installation of well points or larger diameter wells would also encounter difficulties, due to the cobbles and boulders, and would probably require, at a minimum, a duplex rotary percussion drilling system to install the filtered well points with filter packs to the proper depth.

To provide a stable working surface the water level must be controlled to below the base of excavation to avoid potentially piping and disturbance of the subgrade. When wet, silty/sandy subgrades can become easily disturbed, and can lose a significant portion of its natural bearing capacity.

A cofferdam, constructed of earth fill, sand bags, or water filled bag (i.e. aquadam) can be considered at this site. Steel sheet piles with sufficiently robust cross section may also be considered for controlling stream flow, however, the presence of cobbles and boulders in the native soils may limit the penetration of a steel sheet pile type cofferdam. As such, earth fill, sand bags, or water filled bag(s) (i.e. aquadam), are considered more feasible at this site, provided seepage flowing underneath them are considered and well controlled. For base design, sheet piles should extend a minimum depth below base of proposed excavation equal to the height of water above the base of excavation. Since open excavation is anticipated at this site, a by-pass channel can be used to divert stream flow. Alternatively, considering the triple culverts present at this site, consideration should be given to by-pass pumping through existing culvert(s) while the other culvert(s) is replaced.

Ultimately, the method of excavation, dewatering, and stream flow diversion will be the choice of the contractor; however the importance of maintaining the subgrade in a dewatered stable condition during excavation and construction operations cannot be stressed enough.

5.8 CONSTRUCTION CONCERNS

Considering the nature of the embankment fill, no major construction concerns are anticipated if construction is carried out in general conformance with the above discussion. However, cobble and boulder size rock pieces was encountered in the embankment fill and the native soils. The Contractor must be prepared to excavate and install the temporary protection as well as dewater systems considering these materials. If the Sheet Pile Wall is considered for construction, an additional pile load testing will be required if the “refusal” of the production

sheet piles is encountered prior to reaching the required founding level during driving the working sheet piles. A special provision addressing this issue is included in Appendix 5. As noted in Section 5.7 the culvert subgrade must be adequately dewatered to maintain the bearing resistance of the foundation subgrade. The seasonal and yearly fluctuations of the groundwater and the surface water shall be considered for excavation and construction.

6 STATEMENT OF LIMITATIONS

The design recommendations given in this geotechnical report are applicable only to the project described in the text and only if constructed substantially in accordance with details of alignment and elevations stated in the report. Since all details of the design may not be known, in our analysis certain assumptions had to be made. The actual conditions may however, vary from those assumed, in which case changes and modifications may be required to our geotechnical recommendations. We recommend, therefore, that we be retained and provided the opportunity during the design stage to review the design drawings, site survey information, proposed elevations, etc. to verify that they are consistent with our recommendations or the assumptions made in our analysis. It is further recommended that we be retained to review the final design drawings and specifications relative to the geotechnical recommendations.

If, during construction, conditions in the field vary from those assumed at the design stage, an engineer from this office must be notified immediately.

Proper subgrade preparation, groundwater control, compaction, etc. are all critical aspects of the bearing capacity of native soils. It must be noted that different aspects of the geotechnical design are based on the assumption that LVM-Merlex will be retained during site preparation and construction of the proposed works to ensure that both the geotechnical site characteristics and the construction operations/techniques are consistent with our recommendations. Should LVM-Merlex not be involved during the full construction phase, our liability is strictly limited to the factual information contained herein only.

The comments in this report are intended solely for the guidance of the design engineer and address the geotechnical conditions only. The number of boreholes required to determine the localized conditions between boreholes directly affecting construction costs, equipment, scheduling, etc. would in fact be greater than what has been carried out for design purposes. Therefore, contractors bidding on this project or undertaking this work should make their own interpretations of the factual borehole results and carry out further work as they deem necessary to assess the scope of the project.

Section 5 of this reported is intended for the use of the client and the design team only and is not intended to be included in the tender documents. Inclusion of the factual information (Sections 1 to 5 inclusive) in the tender documents is furnished merely for the general information of bidders and is not in any way warranted or guaranteed by or on behalf of the owner or the owner's consultants and its subconsultants or the consultants' or subconsultants' employees, and neither the owner nor its consultants or its employees shall be liable for any representations negligent or otherwise contained in the documents.

Appendix 1 Key Plan

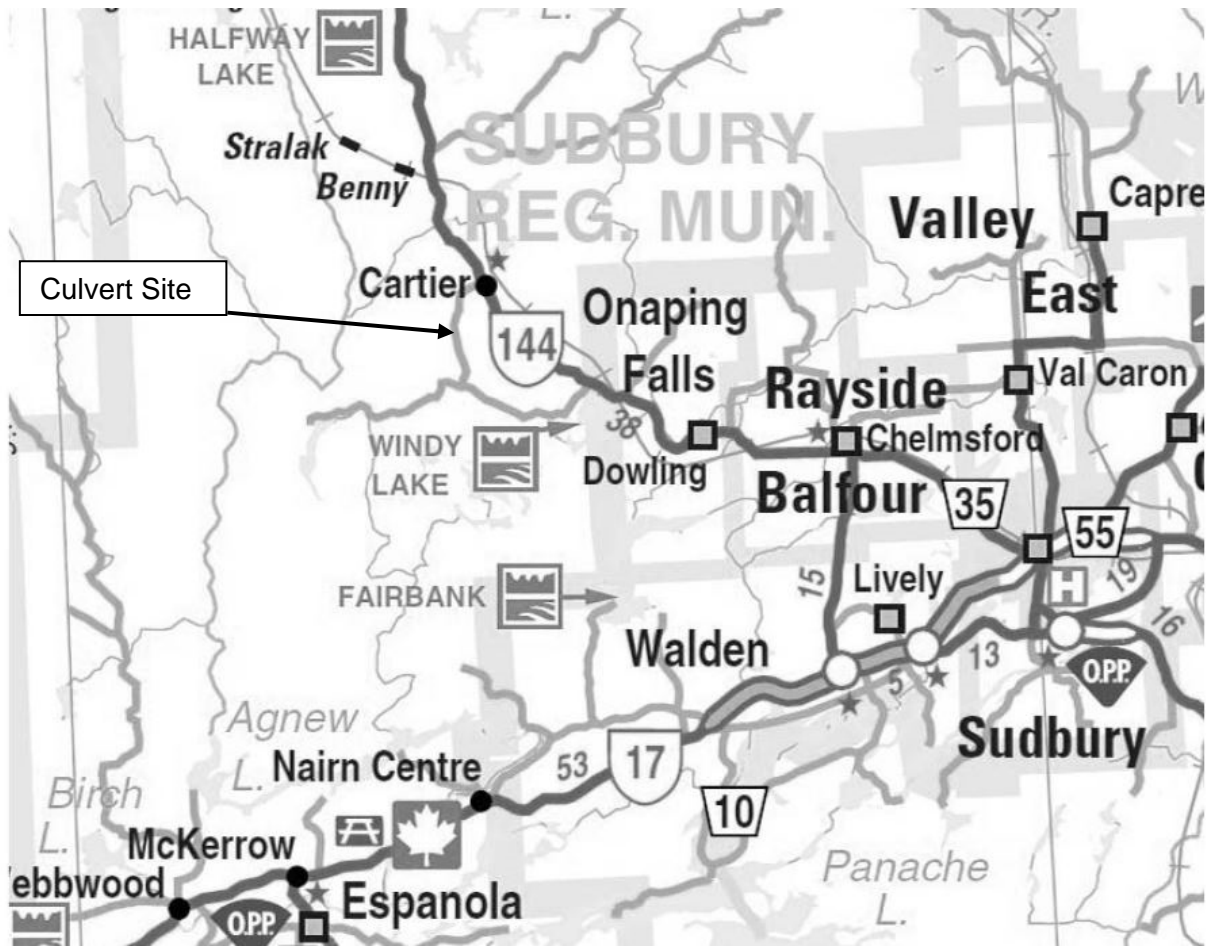
Drawing No. 1

Key Plan

KEY PLAN

Drawing No. 1

NOT TO SCALE



**FINAL
FOUNDATION INVESTIGATION
AND DESIGN REPORT**
GWP 5119-12-00
Highway 7044
Site No. 46-390



Reference No: 11/11/11209

December 2015

Appendix 2 Subsurface Data

Enclosure No. 1	List of Abbreviations and Symbols
Enclosure Nos. 2 to 4	Record of Borehole Sheet
Enclosure Nos. 5 to 6	Record of Testpit Logs

LIST OF ABBREVIATIONS & DESCRIPTION OF TERMS

The abbreviations and terms, used to describe retrieved samples and commonly employed on the borehole logs, on the figures and in the report are as follows:

1. ABBREVIATIONS

AS	Auger Sample
CS	Chunk Sample
DS	Denison type sample
FS	Foil Sample
NFP	No Further Progress
PH	Sampler advanced by hydraulic pressure
PM	Sampler advanced by manual pressure
RC	Rock core with size & percentage of recovery
SS	Split Spoon
ST	Slotted Tube
TO	Thin-walled, open
TP	Thin-walled, piston
WS	Wash Sample
Rec	% recovery from individual run of rock core
RQD	Rock quality designation (%)

2. PENETRATION RESISTANCE/"N"

Dynamic Cone Penetration Test (DCPT):

A continuous profile showing the number of blows for each 300 mm of penetration of a 50 mm diameter 60° cone attached to AW rod driven by a 63 kg hammer falling 760 mm.

Plotted as —●—●—●—●—

Standard Penetration Test (SPT) or "N" Values

The number of blows of a 63 kg hammer falling 760 mm required to advance a 50 mm O.D. drive open sampler 300 mm.

3. SOIL DESCRIPTION

a) *Cohesionless Soils:*

"N" (blows/0.3 m)	Relative Density
0 to 4	very loose
4 to 10	loose
10 to 30	compact
30 to 50	dense
over 50	very dense

b) *Cohesive Soils:*

Undrained Shear Strength (kPa)	Consistency
Less than 12	very soft
12 to 25	soft
25 to 50	firm
50 to 100	stiff
100 to 200	very stiff
over 200	hard

3. SOIL DESCRIPTION (Cont'd)

c) *Cohesive Soils:*

RQD (%)	Classification
Less than 25	Very poor quality
25 to 50	Poor quality
50 to 75	Fair quality
75 to 90	Good quality
90 to 100	Excellent quality

d) *Method of Determination of Undrained Shear Strength of Cohesive Soils:*

- + 3.2 - Field Vane test in borehole.
The number denotes the sensitivity to remoulding.
- D - Laboratory Vane Test
- " - Compression test in laboratory

For a saturated cohesive soil the undrained shear strength is taken as one-half of the undrained compressive strength.

e) *Soil Moisture:*

Moisture	Described as
Dry	Below optimum moisture content
Moist	Near optimum moisture content
Wet	Above optimum moisture content

4. TERMINOLOGY

Terminology used for describing soil strata is based on the proportion of individual particle sizes present in the samples (please note that, with the exception of those samples subject to a grain-size analysis, all samples were classified visually and the accuracy of visual examination is not sufficient to determine exact grain sizing):

Trace, or occasional	Less than 10%
Some	10 to 20%
With	20 to 30%
Adjective (i.e. silty or sandy)	30 to 40%
And (i.e. sand and gravel)	40 to 60%

Terminology for cobbles and boulders is based on auger response and field observations:

Occasional	Obstructions encountered in borehole, however advance is not impeded
Numerous	Obstructions are essentially continuous over drilled length

SAMPLE DESCRIPTION NOTES:

1. **FILL:** The term fill is used to designate all man-made deposits of natural soil and/or waste materials. The reader is cautioned that fill materials can be very heterogeneous in nature and variable in depth, density and degree of compaction. Fill materials can be expected to contain organics, waste materials, construction materials, shot rock, rip-rap, and/or larger obstructions such as boulders, concrete foundations, slabs, abandoned tanks, etc.; none of which may have been encountered in the borehole. The description of the material penetrated in the borehole therefore may not be applicable as a general description of the fill material on the site as boreholes cannot accurately define the nature of fill material. During the boring and sampling process, retrieved samples may have certain characteristics that identify them as 'fill'. Fill materials (or possible fill materials) will be designated on the Borehole Logs. If fill material is identified on the site, it is highly recommended that testpits be put down to delineate the nature of the fill material. However, even through the use of testpits defining the true nature and composition of the fill material cannot be guaranteed. Fill deposits often contain pockets or seams of organics, organically contaminated soils or other deleterious material that can cause settlement or result in the production of methane gas. It should be noted that the origins and history of fill material is frequently very vague or non-existent. Often fill material may be contaminated beyond environmental guidelines and the material will have to be disposed of at a designated site (i.e. registered landfill). Unless requested or stated otherwise in this report, fill material on this site has not been tested for contaminants however, environmental testing of the fill material can be carried out at your request. Detection of underground storage tanks cannot be determined with conventional geotechnical procedures.
2. **TILL:** The term till indicates a material that is an unstratified, glacial deposit, heterogeneous in nature and, as such, may consist of mixtures and pockets of clay, silt, sand, gravel, cobbles and/or boulders. These heterogeneous deposits originate from a geological process associated with glaciation. It must be noted that due to the highly heterogeneous nature of till deposits, the description of the deposit on the borehole log may only be applicable to a very limited area and therefore, caution must be exercised when dealing with a till deposit. When excavating in till, contractors may encounter cobbles/boulders or possibly bedrock even if they are not indicated on the borehole logs. It must be appreciated that conventional geotechnical sampling equipment does not identify the nature or size of any obstruction.
3. **BEDROCK:** Auger refusal may be due to the presence of bedrock, but possibly could also be due to the presence of very dense underlying deposits, boulders or other large obstructions. Auger refusal is defined as the point at which an auger can no longer be practically advanced. It must be appreciated that conventional geotechnical sampling equipment does not differentiate between nature and size of obstructions that prevent further penetration of the boring below grade. Bedrock indicated on the borehole logs will be labeled 'possibly' or 'probable' etc. based on the response of the boring and sampling equipment, surrounding topography, etc. Bedrock can be proven at individual borehole locations, at your request, by diamond core drilling operations or, possibly, by testpits. It must also be appreciated that bedrock surfaces can be, and most times are, very erratic in nature (i.e. sheer drops, isolated rock knobs, etc.) and caution must be used when interpreting subsurface conditions between boreholes. A bedrock profile can be more accurately estimated, at the clients' request, through a series of closely positioned unsampled auger probes combined with core drilling.
4. **GROUNDWATER:** Although the groundwater table may have been encountered during this investigation and the elevation noted in the report and/or on the record of boreholes, it must be appreciated that the elevation of the groundwater table will fluctuate based upon seasonal conditions, localized changes, erratic changes in the underlying soil profile between boreholes, underlying soil layers with highly variable permeabilities, etc. These conditions may affect the design and type and nature of dewatering procedures. Cave-in levels recorded in borings give a general indication of the groundwater level in cohesionless soils however, it must be noted that cave-in levels may also be due to the relative density of the deposit, drilling operations etc.

METRIC**RECORD OF BOREHOLE NO. 1**

REFERENCE 11/11/11209 DATUM Geodetic LOCATION Station 10+003, Twp. of Hart ORIGINATED BY JL/AT
 PROJECT GWP 5119-12-00, Highway 7044 BOREHOLE TYPE Truck Mounted CME 45 - Hollow Stem Augers COMPILED BY AT
 CLIENT Triton Engineering DATE (Started) 1 December 2014 TIME (Completed) 3:55:00 PM CHECKED BY MAM
 DATE (Completed) 3 December 2014

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	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100					
98.1	Ground Surface																
0.0	FILL - gravelly sand to sand and gravel, trace silt brown cobbles encountered (dense)		1	AS												41 50 (9)	
			2	SS	25/25 mm												
			3	SS	35												
96.0	GRAVELLY SAND - trace silt brown, wet cobbles/boulders encountered (compact/dense)		4	SS	47												
2.1			5	SS	79/225 mm											38 54 (8)	
	start advancing casing at 3.7 m depth		6	SS	43												
			7	SS	22												
			8	SS	40												
91.1	SAND - trace gravel trace silt brown, wet (loose/compact)		9	SS	8											2 96 (2)	
7.0			10	SS	29												
88.2	Continued Next Page																

COMMENTS		WATER LEVEL RECORDS	
+ 3, × 3 : Numbers on right refer to Sensitivity Numbers on left refer to values greater than 120 kPa ○ 3% STRAIN AT FAILURE		Date (dd/mm/yy)/Time	Water Depth (m)
		1) 1/12/14 3:50:00 PM	1.1
		2) 3/12/14 3:57:00 PM	1.2
		3)	-

The stratification lines represent approximate boundaries. The transition may be gradual.

MEL-GEO 11209 - BOREHOLE LOGS.GPJ MEL-GEO.GDT 26/5/15

METRIC**RECORD OF BOREHOLE NO. 1**

REFERENCE 11/11/11209 DATUM Geodetic LOCATION Station 10+003, Twp. of Hart ORIGINATED BY JL/AT
 PROJECT GWP 5119-12-00, Highway 7044 BOREHOLE TYPE Truck Mounted CME 45 - Hollow Stem Augers COMPILED BY AT
 CLIENT Triton Engineering DATE (Started) 1 December 2014 TIME (Completed) 3:55:00 PM
 DATE (Completed) 3 December 2014 CHECKED BY MAM

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	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE			"N" VALUES	20	40	60	80					
9.9	GRAVELLY SAND - trace silt brown, wet		11	RC												
87.4	cored through 150 mm cobble at a 10.1 m depth		12	SS	25/0 mm											
10.7	Auger Refusal Unable to further advance casing															

MEL-GEO 11209 - BOREHOLE LOGS.GPJ MEL-GEO.GDT 26/5/15

METRIC**RECORD OF BOREHOLE NO. 2**

REFERENCE 11/11/11209 DATUM Geodetic LOCATION Station 9+996, Twp. of Hart ORIGINATED BY JP
 PROJECT GWP 5119-12-00, Highway 7044 BOREHOLE TYPE Truck Mounted CME 45 - Hollow Stem Augers COMPILED BY AT
 CLIENT Triton Engineering DATE (Started) 10 December 2014 TIME (Completed) 3:05:00 PM
 DATE (Completed) 11 December 2015 CHECKED BY MAM

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	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100					
98.0	Ground Surface																
0.0	FILL - gravelly sand to sand and gravel, trace silt brown, dry cobbles encountered (compact)		1	AS													
			2	SS	17												
			3	SS	15												
95.8	SAND AND GRAVEL - gravelly sand to sandy gravel, trace silt brown, wet cobbles/boulders encountered (compact/dense)		4	SS	27												
2.2			5	SS	16												
			6	SS	17												
			7	SS	37												
	Start advancing casing at 5.5 m depth		8	SS	16												
90.8	SAND - trace gravel trace to some silt brown, wet (loose/compact)		9	SS	7												
7.2			10	SS	11												
	Continued Next Page																

COMMENTS		WATER LEVEL RECORDS	
+ 3, x 3 : Numbers on right refer to Sensitivity Numbers on left refer to values greater than 120 kPa ○ 3% STRAIN AT FAILURE		Date (dd/mm/yy)/Time	Water Depth (m)
		1) 10/12/14 1:55:00 PM	1.4
		2) 11/12/14 7:45:00 AM	1.3
		3)	-

MEL-GEO 11209 - BOREHOLE LOGS.GPJ MEL-GEO.GDT 26/5/15

METRIC**RECORD OF BOREHOLE NO. 2**

REFERENCE 11/11/11209 DATUM Geodetic LOCATION Station 9+996, Twp. of Hart ORIGINATED BY JP

PROJECT GWP 5119-12-00, Highway 7044 BOREHOLE TYPE Truck Mounted CME 45 - Hollow Stem Augers COMPILED BY AT

CLIENT Triton Engineering DATE (Started) 10 December 2014 TIME

DATE (Completed) 11 December 2015 (Completed) 3:05:00 PM CHECKED BY MAM

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100	w_p	w	w_L		
87.8	Continued from Previous Page																
10.2	GRAVELLY SAND - some silt brown, wet (dense)		11	SS	37												
87																	33 55 (12)
86																	
85.8	End of Sampling End of Borehole		12	SS	25/0 mm												
12.2																	

MEL-GEO 11209 - BOREHOLE LOGS.GPJ MEL-GEO.GDT 26/5/15

REFERENCE	11/11/1209	DATUM	Geodetic	LOCATION	Station 9+990, Twp. of Hart	ORIGINATED BY	BT
PROJECT	GWP 5119-12-00, Highway 7044			BOREHOLE TYPE	Track Mounted CME 45 - Hollow Stem Augers	COMPILED BY	AT
CLIENT	Triton Engineering			DATE (Started)	15 December 2014	TIME (Completed)	12:14:00 PM
				DATE (Completed)	16 December 2014	CHECKED BY	MAM

MEL-GEO 11209 - BOREHOLE LOGS.GPJ MEL-GEO.GDT 26/5/15

METRIC**RECORD OF BOREHOLE NO. 3**

REFERENCE 11/11/11209 DATUM Geodetic LOCATION Station 9+990, Twp. of Hart ORIGINATED BY BT

PROJECT GWP 5119-12-00, Highway 7044 BOREHOLE TYPE Track Mounted CME 45 - Hollow Stem Augers COMPILED BY AT

CLIENT Triton Engineering DATE (Started) 15 December 2014 TIME (Completed) 12:14:00 PM CHECKED BY MAM

DATE (Completed) 16 December 2014

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	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100					
	Continued from Previous Page																
85.0			11	SS	22												
11.7	GRAVELLY SAND - some silt cobbles/boulders encountered																
84.3			12	SS	50/80 mm												
12.4	End of Sampling End of Borehole																

MEL-GEO 11209 - BOREHOLE LOGS.GPJ MEL-GEO.GDT 26/5/15

REFERENCE	11/11/11209	DATUM	Geodetic	LOCATION	Station 10+013, Twp. of Hart	ORIGINATED BY	JP
PROJECT	GWP 5119-12-00, Highway 7044			BOREHOLE TYPE	CAT 330 Tracked Excavator	COMPILED BY	AT
CLIENT	Triton Engineering			DATE (Started)	11 December 2014	TIME (Completed)	8:15:00 AM
				DATE (Completed)	11 December 2014	CHECKED BY	MAM

MEL-GEO 11209 - BOREHOLE LOGS.GPJ MEL-GEO.GDT 26/5/15

METRIC**RECORD OF TESTPIT NO. 2**

REFERENCE 11/11/11209 DATUM Geodetic LOCATION Station 9+993, Twp. of Hart ORIGINATED BY JP
 PROJECT GWP 5119-12-00, Highway 7044 BOREHOLE TYPE CAT 330 Tracked Excavator COMPILED BY AT
 CLIENT Triton Engineering DATE (Started) 11 December 2014 TIME (Completed) 9:15:00 AM
 DATE (Completed) 11 December 2014 CHECKED BY MAM

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100	w_p	w	w_L		
97.4	Ground Surface																
0.0	GRAVELLY SAND - some silt some organics, rootlets brown cobbles encountered																
96.4																	
1.0	GRAVELLY SAND - some silt cobbles encountered (<300 mm diameter) grey dense																
92.9																	
4.5	End of Testpit																
COMMENTS							$+3, \times 3$: Numbers on right refer to Sensitivity Numbers on left refer to values greater than 120 kPa ○ 3% STRAIN AT FAILURE					WATER LEVEL RECORDS					
												Date (dd/mm/yy)/Time Water Depth (m) Cave In (m) 1) - ▽ - ▽ 2) - ▽ - ▽ 3) - ▽ - ▽					

The stratification lines represent approximate boundaries. The transition may be gradual.

LVM-Merlex, a Division of EnGlobe Corp.

120 Progress Court, North Bay, On P1A 0C2 Phone: (705)476-2550 Fax: (705)476-8882 Email: northbay@lvm.ca

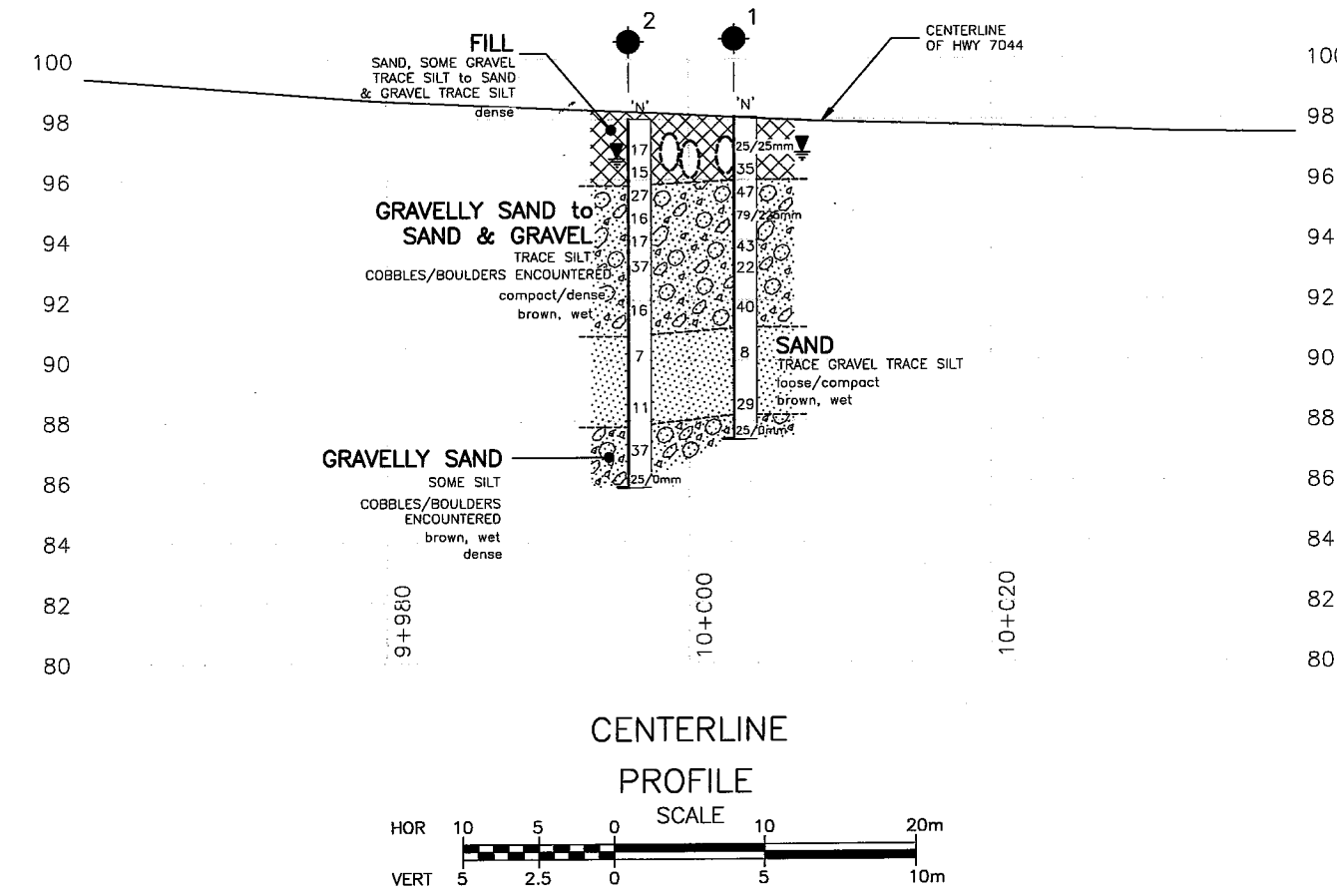
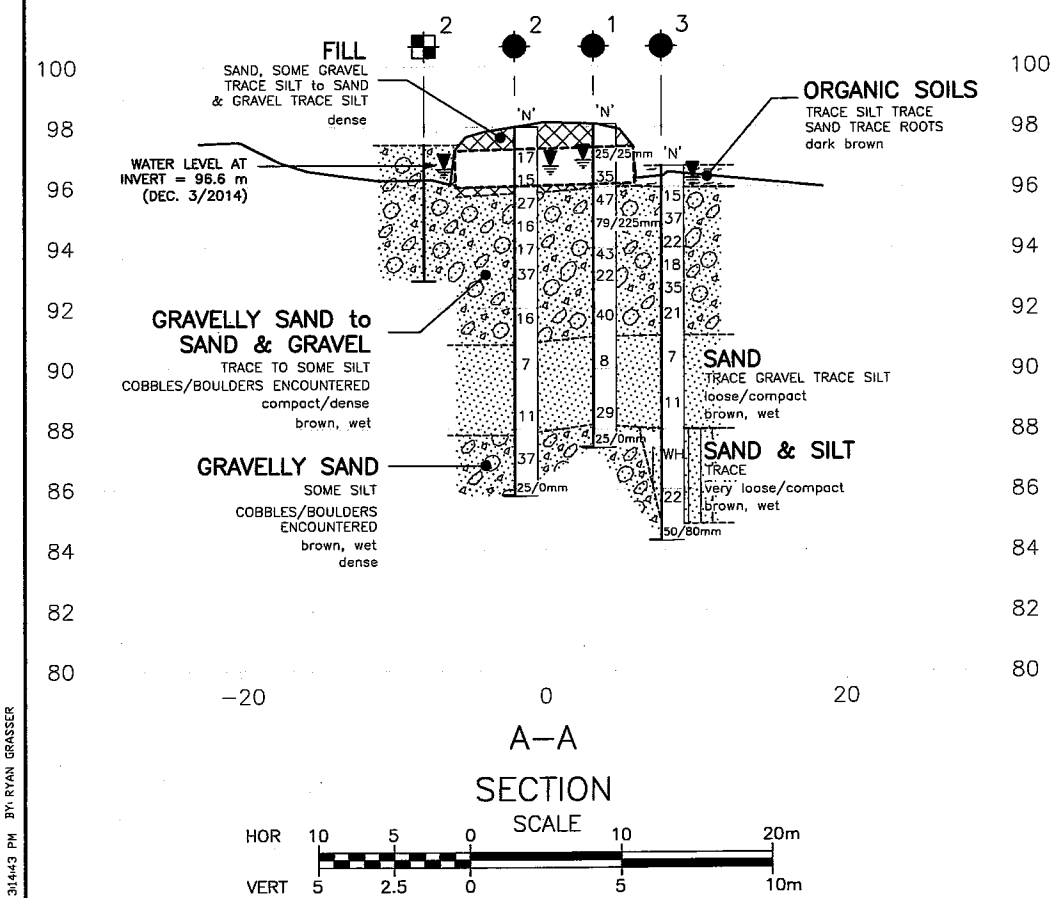
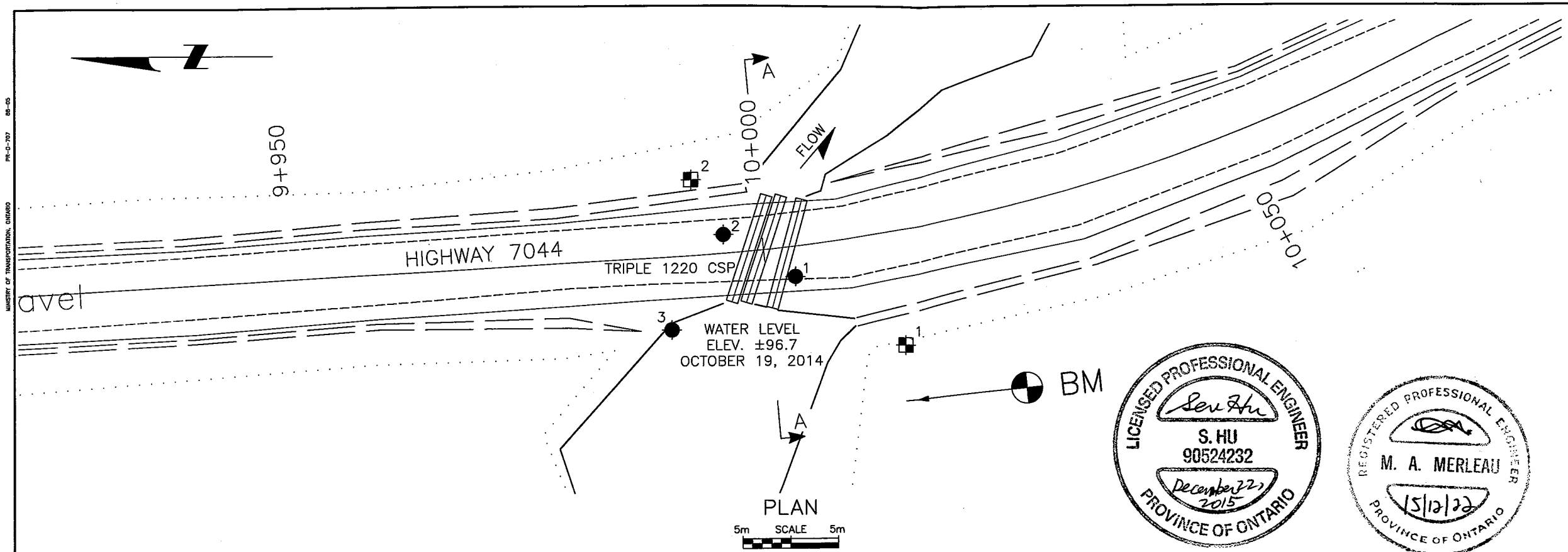
MEL-GEO 11209 - BOREHOLE LOGS.GPJ MEL-GEO.GDT 26/5/15

Appendix 3 Borehole Plan and Lab Data

Drawing No. 2: Borehole Location and Soil Strata
Figure Nos. L-1 to L-5: Grain Size Distribution Curves
Figure No. L-6: Lab Test Summary Sheet

CAD FILE LOCATION AND NAME: E:\2011\11209 - PAVE & FDN & HWY 144 GWP 193-92-00, 5081-06-00 (Triton)\FOUNDATION-FOUNDATION Add Site 46-390\Drawings\Working - Do Not Move or Delete Files\11209 - Borehole Location Plan (FINAL). Site 46-390.dwg
MODIFIED: 12/16/2015 3:05:05 PM BY: GRASBY
DATE PLOTTED: 12/16/2015 3:44:43 PM BY: RYAN GRASSER

MINISTRY OF TRANSPORTATION, ONTARIO
PR-0-707 DB-05



DISTRICT
CONT. No.
GWP No. 5119-12-00

DRAWING
2

HIGHWAY 7044
SITE NO. 46-390
HART TOWNSHIP

BOREHOLE LOCATIONS
AND SOIL STRATA

LVM Merlex

METRIC

LEGEND

●

Borehole

⊕

Testpit

N

Blows/0.3 m (Std Pen Test, 475 J/blow)

DCPT

Blows/0.3 m (60' Cone, 475 J/blow)

▽

Water Level at Time of Investigation

A/R

Auger Refusal at Elevation

E/S

End of Sampling

⬆

Piezometer

BOREHOLE No.	ELEVATION	STATION	O/S
1	98.1	10+003	3.2m Rt
2	98.0	9+996	2.0m Lt
3	96.7	9+990	7.7m Lt

TESTPIT No.	ELEVATION	STATION	O/S
1	97.3	10+013	12.0m Rt
2	97.4	9+993	8.0m Lt

NOTES:

The boundaries between soil strata have been established at the borehole locations only. The boundaries illustrated and stratigraphy between boreholes on this drawing are assumed based on borehole data and may vary. They are intended for design only.

Base plan and alignment provided in digital format by exp. on December 10, 2014.

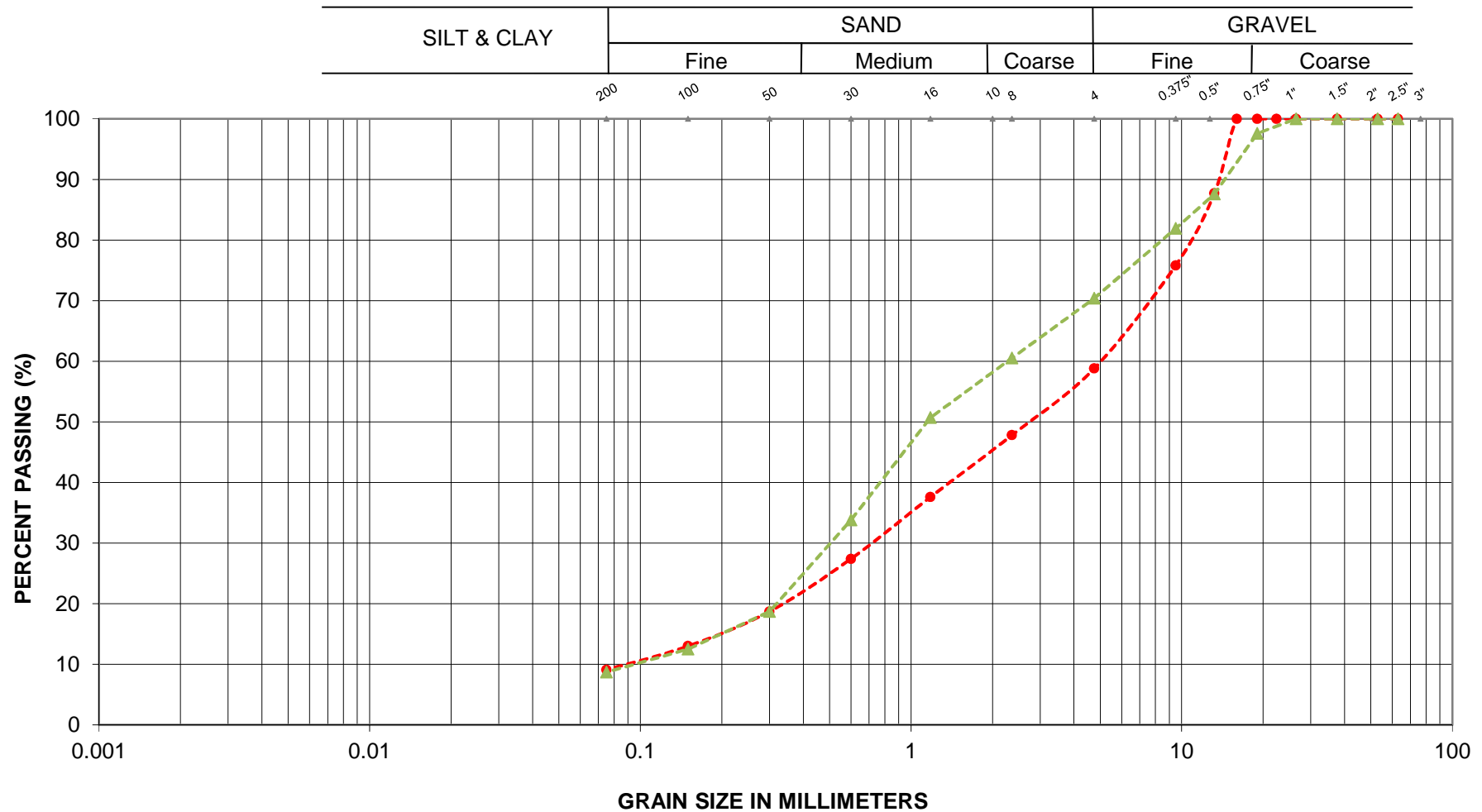
GEOCRES No. 411-336

This drawing is for subsurface information only. Surface details and features are for conceptual illustration. The proposed structure location is shown for illustration purposes only and may not be consistent with the final design configuration as shown elsewhere in the Contract Documents.

DRAWING NOT TO BE SCALED
50mm ON ORIGINAL DRAWING

APR/15	RG	DRAFT
DEC/15	RG	FINAL
DESIGN		
CHK	CODE	LOAD
DRAWN	RG	CHK AT
SITE 46-390		
STRUCT		
SCHEME		
DATE DEC/15		
DWG 2		

GRAIN SIZE ANALYSIS



--●-- BH No.: 1 Sa No.: 2 Depth: 0.8 - 1.5 m

--▲-- BH No.: 2 Sa No.: 3 Depth: 1.5 - 2.0 m

G.W.P.: 5119-12-00

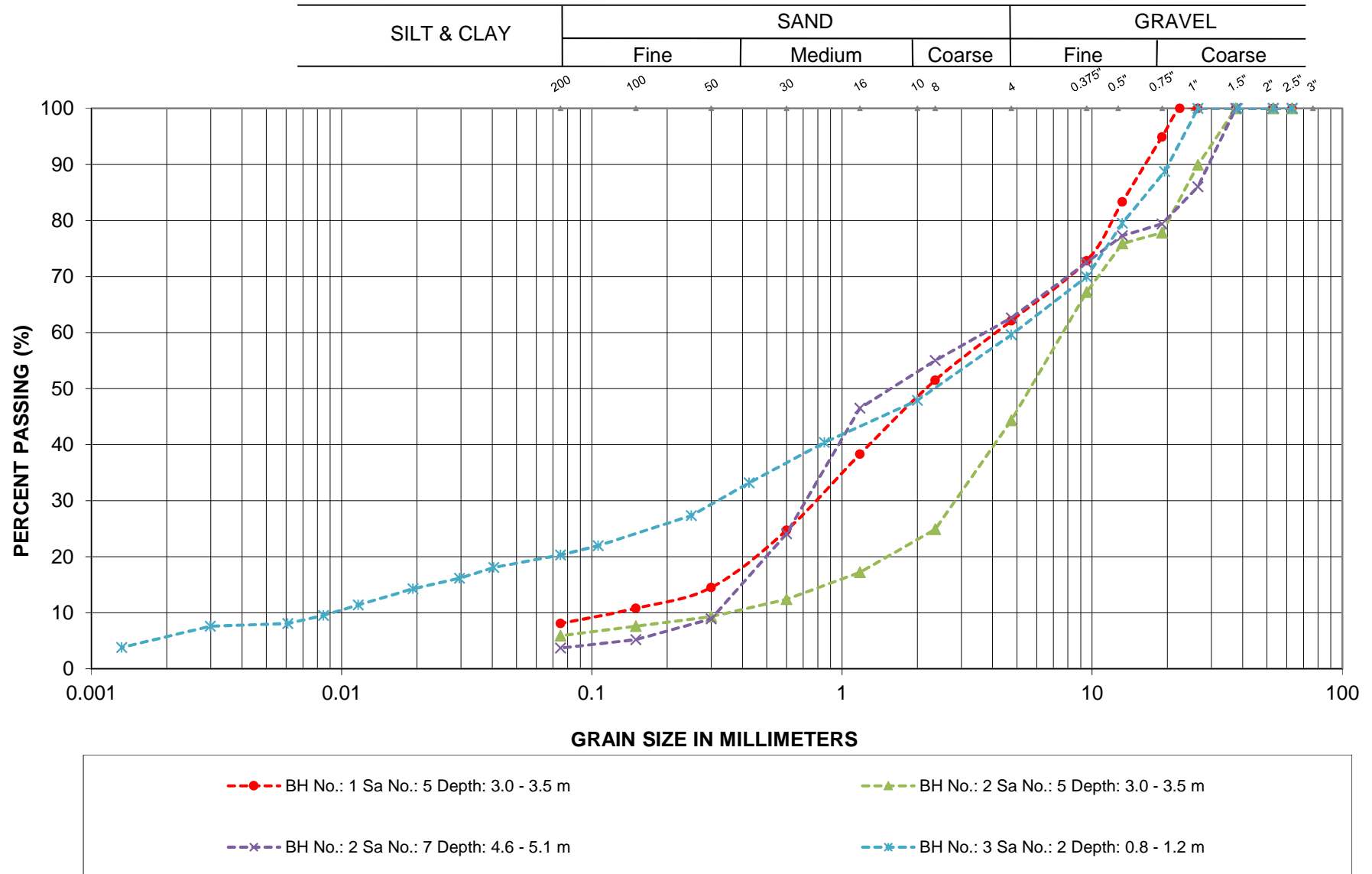
LOCATION: Hwy 7044, Site No. 46-390

FILL

LVM-MERLEX

FIGURE L-1

GRAIN SIZE ANALYSIS



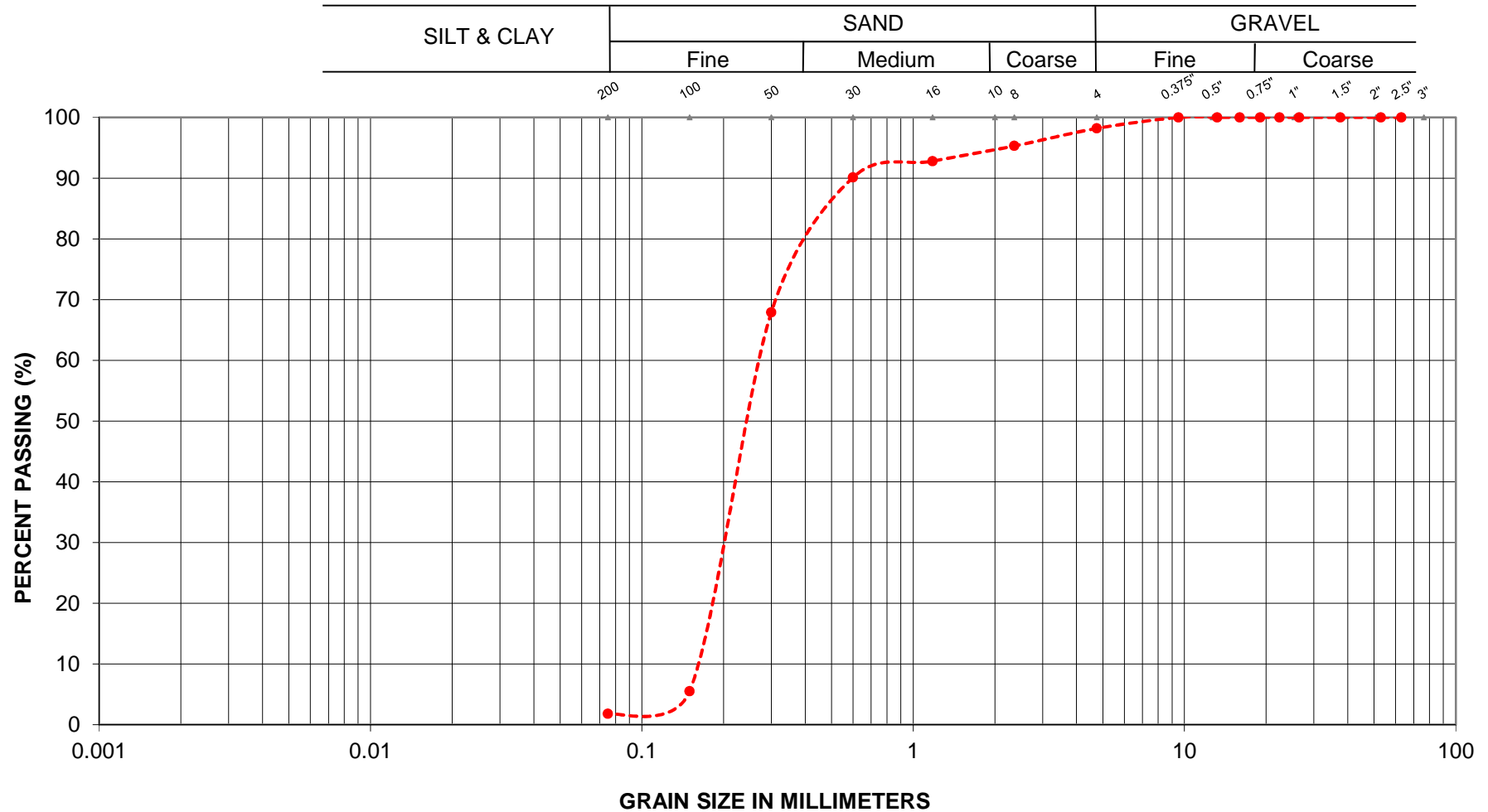
G.W.P.: 5119-12-00
LOCATION: Hwy 7044, Site No. 46-390

SANDS AND GRAVEL

LVM-MERLEX

FIGURE L-2

GRAIN SIZE ANALYSIS



---●--- BH No.: 1 Sa No.: 9 Depth: 7.6 - 8.1 m

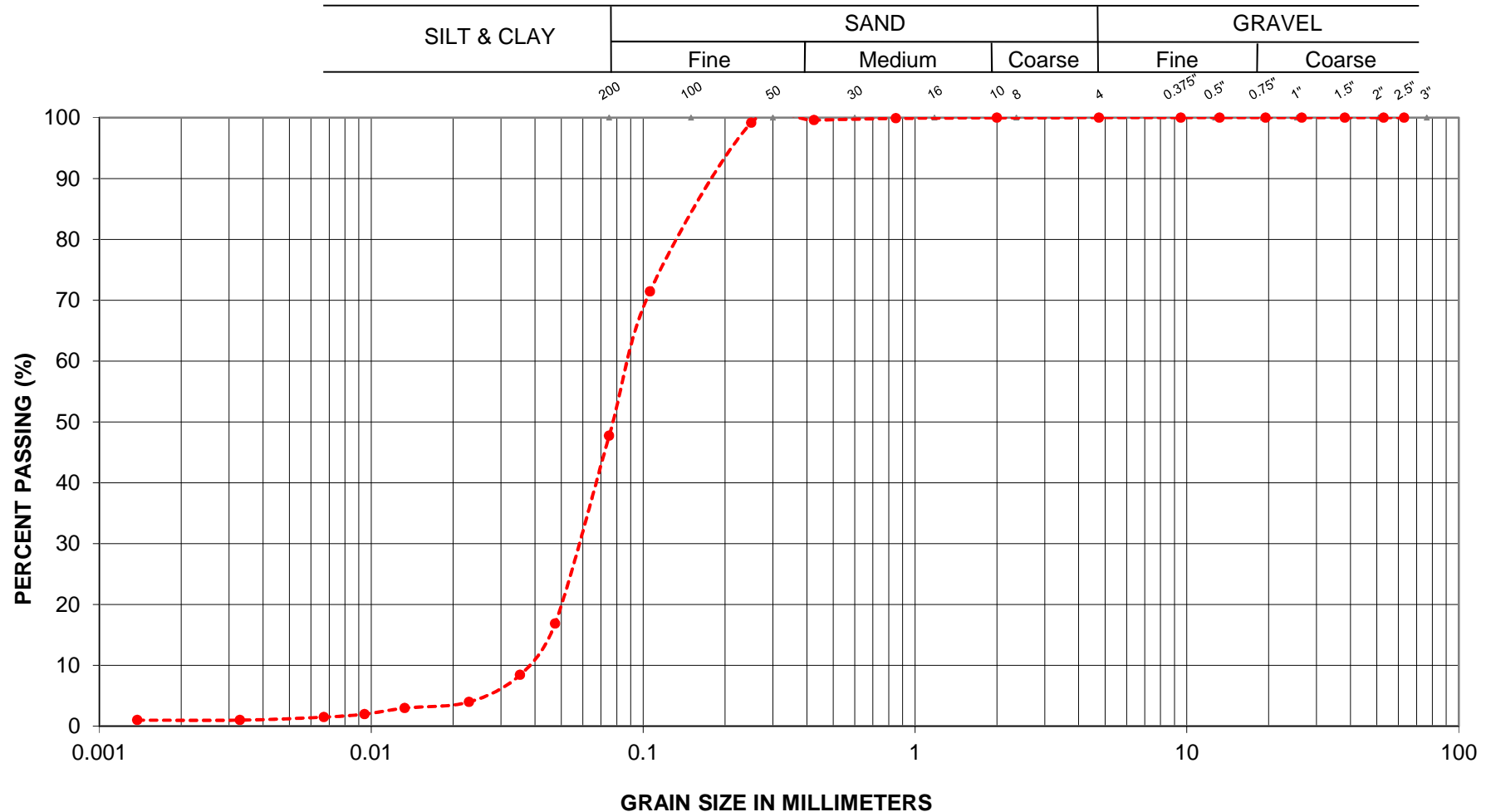
G.W.P.: 5119-12-00
LOCATION: Hwy 7044, Site No. 46-390

SANDS

LVM-MERLEX

FIGURE L-3

GRAIN SIZE ANALYSIS



---●--- BH No.: 3 Sa No.: 10 Depth: 9.1 - 9.6 m

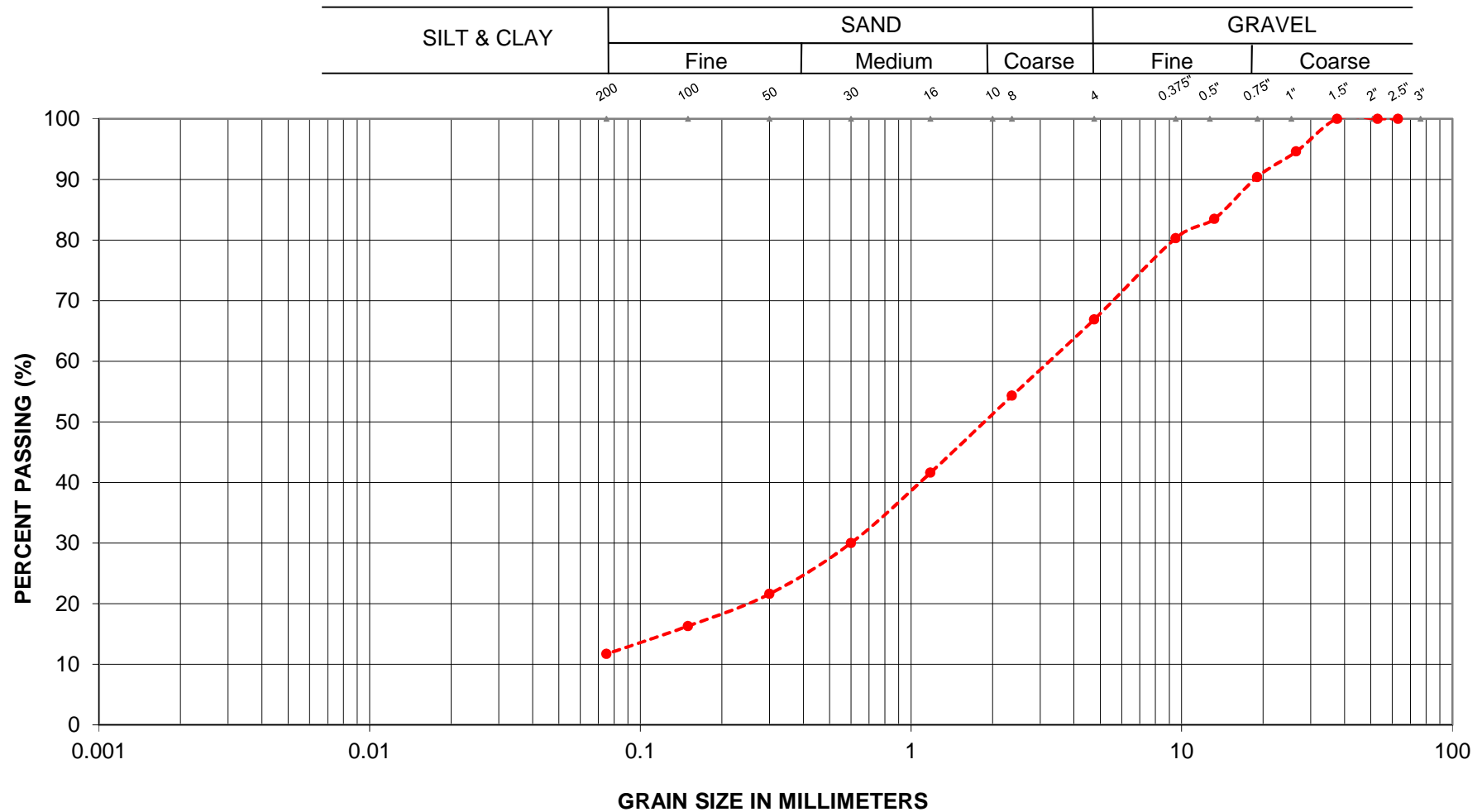
G.W.P.: 5119-12-00
LOCATION: Hwy 7044, Site No. 46-390

SAND AND SILT

LVM-MERLEX

FIGURE L-4

GRAIN SIZE ANALYSIS



---●--- BH No.: 2 Sa No.: 11 Depth: 10.7 - 11.1 m

G.W.P.: 5119-12-00
LOCATION: Hwy 7044, Site No. 46-390

GRAVELLY SAND

LVM-MERLEX

FIGURE L-5

Laboratory Tests - Summary Sheet

Borehole No.	Sample No.	Depth	Grain Size Analysis				NMC	Atterberg Limits			SPT 'N'	USCS	Unit Weight (kN/m ³)	Remarks
			Gravel Size (%)	Sand Size (%)	Silt Size (%)	Clay Size (%)		LL (%)	PL (%)	IP (%)				
1	1	0.0					4.6							
	2	0.8	41	50	9		5.3				25/25 mm			
	3	1.5					13.3				35			
	4	2.3					10.2				47			
	5	3.1	38	54	8		11.9				79/225 mm			
	6	4.1					10.8				43			
	7	4.6					11.4				22			
	8	6.1					13.9				40			
	9	7.6	2	96	2		22.6				8			
	10	9.1					17.6				29			
	11	10.1												Coring
	12	10.7									25/0 mm			
2	1	0.0					2.3							
	2	0.8					2.1				17			
	3	1.5	30	61	9		9.0				15			
	4	2.3					11.2				27			
	5	3.1	56	38	6		12.5				16			
	6	3.8					11.3				17			
	7	4.6	37	59	4		13.7				37			
	8	6.1					18.1				16			
	9	7.6					23.1				7			
	10	9.14					23.4				11			
	11	10.67	33	55	12		11.7				37			
	12	12.2									25/0 mm			

Laboratory Tests - Summary Sheet

[illegible]

Appendix 4 Photo Essay

Enclosure No. 7:

Photo Essay

Existing Embankment – Looking North

Photo: 1



Culvert Outlet – Looking East

Photo: 2



Project: Hwy 7044 – Site No. 46-390

Photos Provided By: LVM

Date: December 2014

Culvert Invert – Looking South. Note: Berm of cobble/boulder size rock

Photo: 3



Testpit No. 2. Note: Cobbles encountered and groundwater in testpit.

Photo: 4



Project: Hwy 7044 – Site No. 46-390

Photos Provided By: LVM

Date: December 2014

Cobbles excavated from Testpit No. 1.

Photo: 5



Culvert Invert – Looking West. Note: Berm of cobble/boulder size rock

Photo: 6



Project: Hwy 7044 – Site No. 46-390

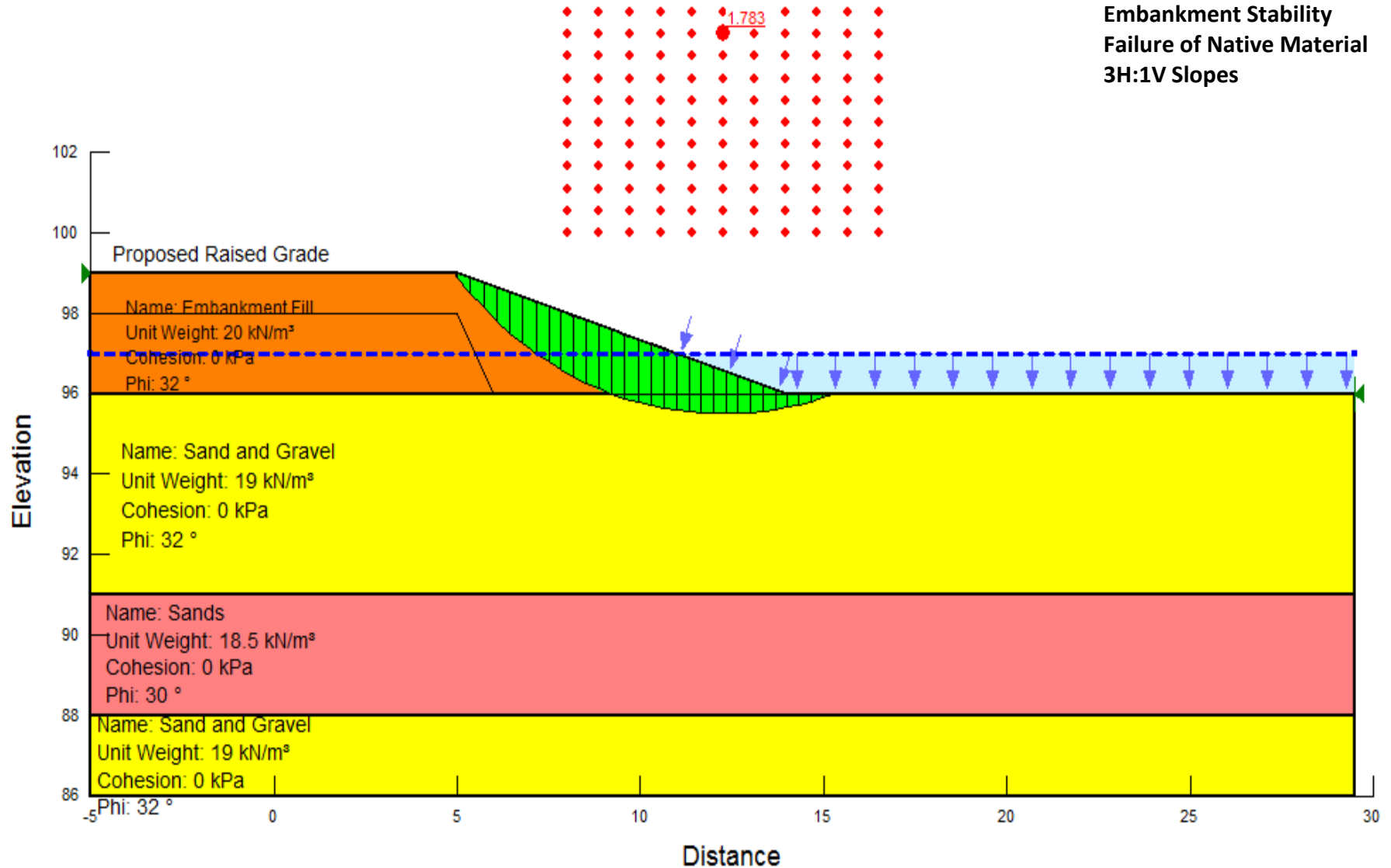
Photos Provided By:
LVM/Google

Date: December 2014

Appendix 5 Design Data

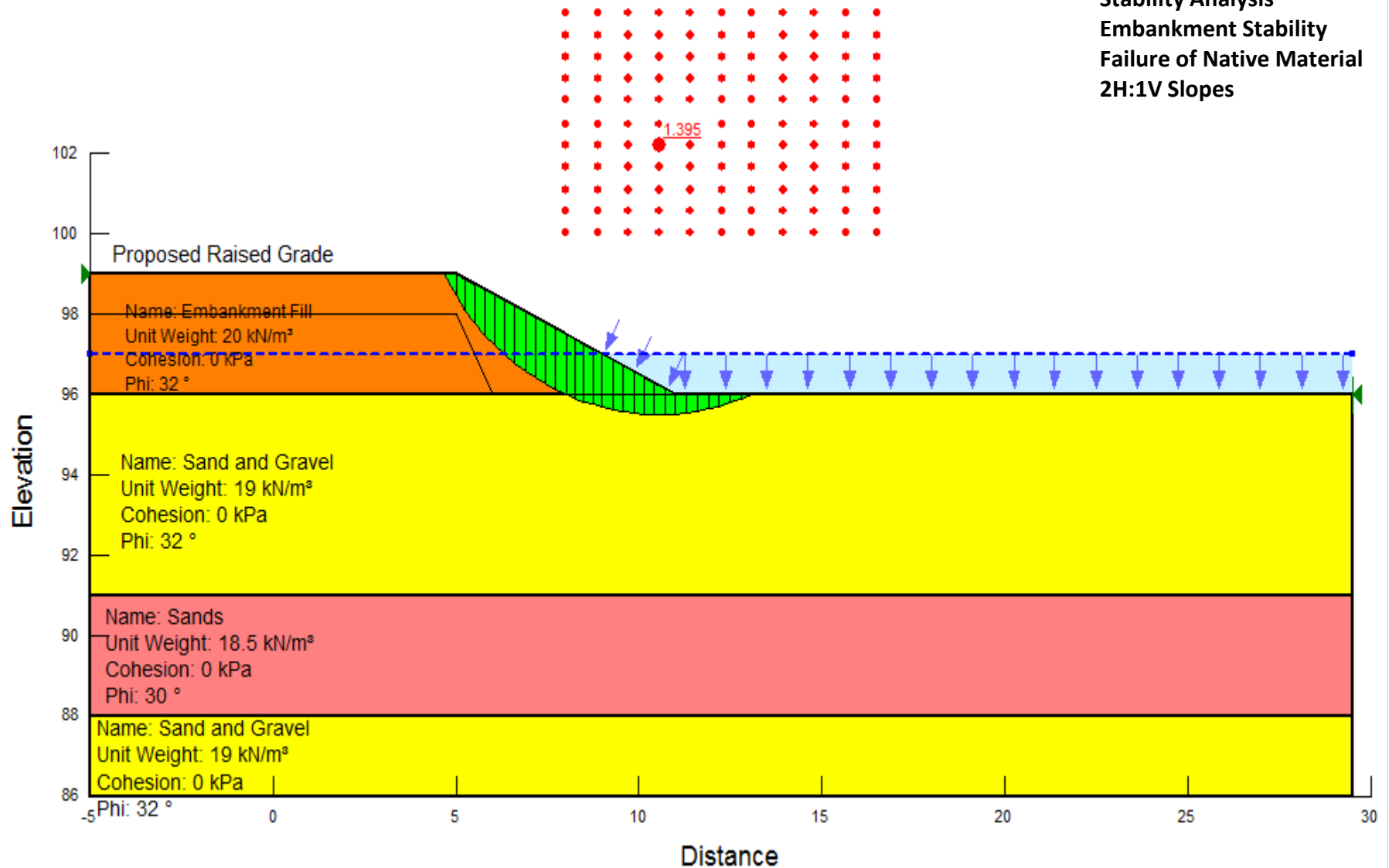
Figure Nos. S-1 and S-2: Slope Stability
Table A: Comparison of Shoring Alternatives
 Notice to Contractor

**Stability Analysis
Embankment Stability
Failure of Native Material
3H:1V Slopes**



Stability Analysis
Culvert 10+000
TWP of Hart

Stability Analysis
Embankment Stability
Failure of Native Material
2H:1V Slopes



Stability Analysis
 Culvert 10+000
 TWP of Hart

Table A – Comparison of Shoring Alternatives

Method	Depth Range (m)	Advantages	Disadvantages	Remarks	Estimated Costs
Wood Sheeting	1.5 – 5	-Low cost, -Easily installed in good ground conditions	-Limited by soil conditions, -Limited depth of installation, -Low strength, -discontinuous	Not considered due to rock fill embankment	\$ 650/m ²
Steel Sheet Piles	5 – 21	-High strength, continuous, -Readily available	-Limited by soil conditions (i.e. obstructions)	Considered on the conditions of sufficiently robust cross section and installation trial testing on site	Not less than \$ 650/m ² , depending on robust cross section
Pre-cast concrete panels	3 – 10	-Durable -Assists in minimizing seepage	-Limited depths -Can be damaged during installation -Limited by soil conditions (i.e. obstructions)	Not considered due to ground conditions and higher cost	
Soldier piles	5 – 25	-Easy installation -Readily available -Adaptable to various ground conditions	-Pre-drilling may be required -Possible ground loss	Considered as alternative on the conditions of dewatering pre-drilling the borehole if cobble/boulder encountered in embankment fills and native soils	\$ 725/m ² Predrilling \$ 1,500/m ²
Tangent/ Secant/ Staggered Drilled Piles	10 – 18	-Readily available -Adaptable to various ground conditions	-Possible ground loss and/or seepage -Poor alignment tolerance	Considered as alternative for excavations requiring a protection system	
Concrete Diaphragm	10 – 30	-High Strength -Durable -Can be permanent	-High cost -Requires specialized equipment/control	Not Considered due to ground conditions and higher costs	
Micropiles with reinforced shotcrete face		-Can be installed in various ground conditions -High strength -Good tolerance	-High Cost -Requires specialized equipment	Considered as alternative for excavations requiring a protection system	\$ 1,200 – 1,500/m ²

NOTICE TO CONTRACTOR – Obstructions in Fills and Native Soils as well as Fluctuations of Ground/Surface Water

Special Provision

The Contractor is notified that, during foundation field investigations for the Culverts at Site No. 46+390, on Highway 7044, cobble/boulder sized rock pieces were encountered in the embankment fills, and native sands and gravels deposits. The contractor shall take into account these materials when designing and installing the temporary Protection and Dewatering Systems. If the Sheet Pile Wall is considered for construction, an additional pile load testing will be required if the “refusal” of the production sheet piles is encountered prior to reaching the required founding level during driving the working sheet piles. The seasonal and yearly fluctuations of the groundwater and the surface water shall be considered for excavation and construction.



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