

**Submitted To AECOM Canada Ltd.
189 Wyld Street Suite 103, North Bay, Ontario P1B 1Z2
On Behalf of the Ontario Ministry of Transportation**

**Culvert Replacement – Boyne Creek Culvert
Highway 60
Station 15+935 – Township of Franklin
Site No. 42-043/C
GWP 5333-11-00**

FINAL PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT

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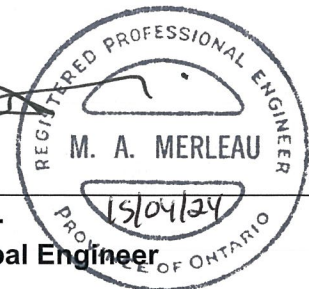


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1 INTRODUCTION

LVM-Merlex, a Division of EnGlobe Corp., has been retained by AECOM Canada Ltd., on behalf of the Ministry of Transportation of Ontario (MTO), to carry out a preliminary foundation investigation at an existing centerline culvert site (Site No. 42-043/C) located at Station 15+935 in the Township of Franklin on Highway 60, some 2.3 km west of the intersection between Highway 60 and Highway 35 (see Drawing No. 1 in Appendix 1).

The foundation investigation location was specified by the MTO in the Terms of Reference for work under Agreement No. 5013-E-0032: GWP 5333-11-00 for Design-Build. The terms of reference for the scope of work are outlined in LVM-Merlex's Proposal P-14-051 dated May, 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

The single span cast-in-place concrete Rigid Frame Box (RFB) culvert is located on Highway 60 at Station 15+935 in the Township of Franklin. At the culvert location, the highway runs locally in a north-south direction. The flow through the culvert is from the west to the east (right to left). A north-south orientation is used in this report for description purposes.

The culvert was constructed in 1963. The structure was rehabilitated in 1964 with the addition of a thickened deck slab at the east end, as well as increased wall thickness on the south side. The concrete RFB culvert at this location has an inside span of 9.14 m in width, a height of 4.57 m, and is some 19 m in length. The culvert has concrete wing walls on all four quadrants approximately 5.8 metres in length at a 30° skew from the culvert barrel.

The topography at this site is located in a valley area. The existing highway, at the culvert location, is constructed through a granular fill embankment some 7.6 m in height, with centerline elevation of 326.8 m at the culvert location. The existing embankment extending out from the existing concrete wing walls, in the area of the culvert, have been built on slope angles of approximately 2H:1V to 2.3H:1V.

2.1 SITE PHYSIOGRAPHY AND SURFICIAL GEOLOGY

This project is located in the Geomorphic Sub-province known as the Muskoka Ridges and Pockets. The topography on this section of Highway 60 is generally rolling. Significant layers of earth overlay the bedrock. Organic materials were also observed. Within the project area native overburden primarily consists of sand and silt overlying the silts overlying the bedrock.

Bedrock in the area consists of the migmatitic rocks and gneisses of undetermined protolith.

3 INVESTIGATION PROCEDURES

The fieldwork for this investigation was carried out during the period between August 26th and September 3rd, 2014 during which time three (3) sampled boreholes were advanced. One (1) borehole was advanced through the embankment at the location of the culvert, and a single borehole was advanced at each of the inlet (west) and the 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. 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 were observed during the advancement of and immediately following, completion of the individual boreholes. A single 19 mm diameter standpipe was installed in one (1) open borehole prior to backfilling to allow for further monitoring of the shallow groundwater levels. 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 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-3 and Table No. L4).

The location of the individual boreholes was determined in the field using highway chainage (established by others) and offset relative to highway centerline. 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. The borehole elevations are based on a survey carried out by others.

4 SUBSURFACE CONDITIONS

Details of the subsurface conditions revealed by the investigation program are presented on the enclosed Records of Borehole Logs (Appendix 2) and on Drawing No. 2 (Appendix 3). Please note that stratigraphic delineations 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 15+935, TOWNSHIP OF FRANKLIN

A plan and profile illustrating the borehole locations and stratigraphic sequences is shown on Drawing No. 2, Appendix 3. During the course of the preliminary exploration program, three (3) sampled boreholes were put down at this site, with Borehole No. 1 advanced at the culvert inlet (right side), Borehole No. 2 advanced through the embankment, and Borehole No. 3 advanced at the culvert outlet (left side). At the time of the subsurface investigation, the ground surface elevations at Boreholes Nos. 1 to 3 were recorded at elevations 322.1 m, 326.5 m, and 323.4 m, respectively.

4.1.1 Pavement Structure

Borehole No. 2 was advanced through the embankment where a pavement structure consisting of 102 mm asphalt and 203 mm crushed gravel was penetrated.

4.1.2 Granular Fill

Underlying the pavement structure at Borehole No. 2 and below ground surface at Borehole No. 1, a layer of granular fill consisting of brown sand with to trace gravel, silty to trace silt, trace clay was penetrated. The natural moisture content measured on samples of this deposit recovered in Borehole No. 2 was in the order of 4% to 18%. On the one sample recovered from Borehole No. 1 the natural moisture content was measured at 58%. Gradation analysis was carried out on one (1) sample of this deposit, the results of which indicated 24% gravel size particles, 69% sand size particles, and 7% silt and clay size particles (Figure No. L-1, Appendix 3). Based on SPT 'N' values of 9 to 21 blows per 300 mm penetration, the compactness of this deposit was described as loose to compact. This deposit was encountered to depths of 0.6 m and 4.4 m below grade at Borehole Nos. 1 and 2 respectively (elevation 321.5 m and 322.1 m, respectively).

4.1.3 Sand and Silt

Underlying the silty sand fill and the sand fill at Borehole Nos. 1 and 2, and below ground surface at Borehole No.3, a layer of brown to grey sand and silt trace gravel trace clay was penetrated. The natural moisture content measured on samples of this deposit was in the order of 17% to 33% except two samples of 52% and 47% encountered at shallow depths of the deposit in Borehole Nos. 1 and 2. Gradation analyses were carried out on five (5) samples of this deposit, the results of which indicated 0% to 9% gravel size particles, 42% to 54% sand size particles, 45% to 57% silt size particles, and 1% to 2% clay size particles (Figure Nos. L-2 in Appendix 3). Based on SPT 'N' values of 1 to 14 blows per 300 mm penetration, this deposit was described as very loose to compact, generally very loose. This deposit was encountered to depths of 2.9 m, 10.1 m, and 10.1 m below grade at Borehole Nos. 1 to 3, respectively (elevations 319.2 m, 316.4 m and 313.3 m, respectively).

4.1.4 Silt

Underlying the sand and silt deposit at Borehole Nos. 1 to 3 inclusive, a layer of grey silt, sandy to trace sand, trace gravel, trace clay was penetrated. The natural moisture content measured on samples of this deposit was in the order of 21% to 29%. Gradation analyses were carried out on three (3) samples of this deposit, the results of which indicated 0% to 6% gravel size particles, 7% to 34% sand size particles, 45% to 82% silt size particles, and 2% to 7% clay size particles (Figure Nos. L-3 in Appendix 3). Based on SPT 'N' values of 4 to 59 blows per 300 mm penetration and 75 blows per 50 mm penetration, this deposit was described as loose to very dense, generally compact. This deposit was encountered to depths of 5.5 m, 14.9 m, and 11.0 m below grade at Borehole Nos. 1 to 3, respectively (elevations 316.6 m, 311.6 m and 312.4 m, respectively), where bedrock was encountered.

4.1.5 Bedrock

Underlying the above described silts at Borehole Nos. 1 to 3 inclusive, the bedrock was proven by diamond core drilling. The bedrock was described as grey to pink gneiss bedrock. Based on Rock Quality Designation (RQD) values ranging from 74% to 91%, the bedrock was described as good to excellent quality. Sampling in the bedrock was terminated at depths of 8.6 m, 18.0 m, and 14.0 m below grade at Borehole Nos. 1 to 3, respectively (elevations 313.5 m, 308.5 m, and 309.4 m, respectively). It should be noted that, when encountered, the underlying bedrock surfaces in this area can be very erratic in nature, varying substantially in elevation over short horizontal distances.

4.2 GROUNDWATER DATA

The survey information indicates that the water level in the creek was measured at some 321.6 m on August 14, 2014 by others. At the time of this investigation, the creek water levels were measured at elevation 321.7 m at the inlet area on August 26th, 2014 and 321.9 m at the outlet area on August 27th, 2014, respectively.

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. A standpipe was installed in Borehole Nos. 1 and 2 to obtain post borehole completion water levels. These levels are recorded on the individual Record of Borehole Log Sheets (Appendix 2) and shown on the Borehole Locations and Soil Strata Drawing No. 2 in Appendix 3.

The water levels were measured at elevations 321.7 m (2014-08-26), 321.8 m (2014-09-04), and 321.8 m (2014-08-27) at Borehole Nos. 1 to 3, respectively.

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

5 DISCUSSION AND RECOMMENDATIONS

5.1 GENERAL

A foundation investigation was carried for the proposed replacement of a concrete rigid frame box (RFB) culvert, Site No. 42-043/C as identified by the MTO.

The existing culvert, located at Station 15+935 in the Township of Franklin, has an inside span of 9.14 m in width, a height of 4.57 m, and is some 19 m in length. The invert of the existing culvert at centerline is at a depth of some 7.4 m (elevation 319.4 m) below centreline. The culvert has wing walls/retaining walls on all four quadrants that are approximately 5.8 metres in length at a 30° skew from the culvert barrel.

The existing highway embankment currently supports two undivided lanes of highway, locally running in a north-south direction at the culvert location. The flow through the existing culvert is from right to left (west to east). Based on data from this foundation investigation, the embankment supporting the existing pavement at this site has been constructed using a granular pavement structure overlying granular fills. The native material, underlying the embankment fill, generally consisted of very loose sand and silt overlying silts overlying bedrock.

The RFB structure was rehabilitated in 1964 with the addition of a thickened deck slab at the north end, as well as increased wall thickness on the west side. Based on the requirements stated in RFP, the existing box culvert will be replaced; however it is understood that a technical memorandum dated November 17th, 2014 and prepared by AECOM indicates that rehabilitation of two culvert ends is recommended instead of culvert replacement. It is understood that MTO indicates that the existing culvert is to have a minor patch-repair treatment which is not being considered as “rehabilitation”; this preliminary Foundation Investigation and Design Report (FIDR) is prepared based upon replacing the culvert in accordance with the requirements stated in RFP.

For the report preparation, it is assumed that the type of the new culvert is to be constructed of reinforced concrete. It is also assumed that the new culvert will be constructed along a similar skew and alignment and the final vertical alignment of the highway will remain essentially the same.

5.2 FOUNDATION CONSIDERATIONS

The founding native very loose sand and silt overlying silts overlying the bedrock present below the existing embankment are considered adequate for support of a box culvert and for a conventional highway embankment of this height. Bearing resistance, for a box type structure, should not be a major issue provided the natural bearing surface is not excessively disturbed during construction and the groundwater level is adequately controlled throughout construction, as discussed in Section 5.5.

Based on the characteristics of the native sand and silt subgrade present below the culvert, the response of the existing embankment, and a founding elevation similar to that of the existing culvert, a factored bearing resistance at ULS of 300 kPa can be used for a rigid frame box culvert (i.e. concrete frame box culvert). In consideration of the width of the culvert (essentially acting as a raft), depth of overburden, and response of the existing embankment, a geotechnical reaction at SLS of 80 kPa can be used for design, in consideration of 25 mm settlement.

Open types of culvert (i.e. concrete rigid frame open culvert with wall footings) are not recommended when considering the required excavation depth for scouring protection, very loose sand and silt subgrade, footing size effect, and the water levels of the creek.

5.2.1 Slope Stability

The maximum height of the embankment above the stream bed at this location is some 4 m on the western side slope and approximately 5 m on the eastern side slope. The inclination angles of the existing slopes are some 1H:1V on the western (right) slope and 1.7H:1V to 2.5H:1V on the eastern (left) slope. Stability analyses, using the GEO-SLOPE computer program, Slope/W (GeoStudio 2007, version 7.17, Geo-Slope International Ltd.), were carried out at this location for the east slope and with existing inclinations and standard embankment slopes of 2H:1V in the granular fill, respectively. For the purposes of these analyses, the materials were modeled using the following parameters;

PARAMETER	MATERIAL		
	EMBANKMENT FILL	SAND AND SILT	SILT
Unit Weight (kN/m ³)	19	18.5	18
Effective Friction Angle (degrees)	30	26	28

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 indicate factors of safety in the order of 1.3 for the 2H:1V side slopes out of the existing wing walls, except a lower value in the order of 1.1 against the minor surficial slippage of the existing embankment (see Figure No. S-1, Appendix 5). 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 constructed.

5.3 CULVERT DESIGN, BEDDING, AND EMBEDMENT

The embankment consists of granular fills. The results of this investigation indicate that, below the culvert invert, the native soils encountered at Boreholes No. 1 to 3 inclusive, consisted of very loose sand and silt overlying loose to compact silt. A review of the condition of the pavement surface, at the culvert location, revealed minor asphalt cracking; however, in general,

the embankment appears to have performed well. The existing embankment has preloaded the soils at the culvert locations and since there will be no change in the height of the embankment, and therefore no increases in embankment load, no appreciable consolidation settlement of the embankment is anticipated. As such, installing the culverts on a camber will not be required at this site.

5.3.1 **Rigid Concrete Box Culvert**

Flexible culverts (i.e. CSP/HDPE) and/or concrete pipes will not be considered for culvert replacement due to the size of existing culvert at this site. Also, as previously noted, open bottom type of culverts will also not be considered due to subgrade conditions. As such, a cast-in-place, or precast concrete rigid frame box culvert can be considered for culvert replacement at this site.

The frost penetration depth of 1.8 m below ground surface without snow cover in the Huntsville area should be considered for the rigid frame box culvert. Bedding for 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 upcompacted. 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 bedding and levelling course consisting of 19 mm clear stone per OPSS.PROV 1004 should be used, which would aid in dewatering applications. During backfilling, the material of bedding, cover and backfill shall be placed in uniform layers not exceeding uncompacted thickness of 200 mm. Backfilling shall be placed in a balanced manner in layers not exceeding 200 mm in thickness on each side of the box unit. The elevation difference of backfilling on either side of the box unit shall be limited to a maximum 400 mm as per OPSS 422. Backfill below the pavement structure and within the depth of frost penetration should be at a minimum granular material of Select Subgrade Material (SSM) or better per OPS.PROV 1010. 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 with a strip of Non-Woven Class II Geotextile (per OPSS 1860) 600 mm in width, centered over the joint, covering the top of the culvert and extending 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 rip-rap (R-50 size as per OPSS.PROV 1004) apron. The apron shall be 5 m in length, 400 mm thick, and extend across the stream bed to 5 m beyond the outside edges of the culvert. Clay seals are generally used where significant head differences exist between the inlet and outlet of the culverts to prevent flow through the bedding/embedment granulars. Considering the head difference between the inlet

and outlet and subsurface conditions, clay seals are not considered necessary at this culvert location.

Alternatively the concrete wingwalls can be considered for the stream bed protection, similar to the existing structure.

5.4 CULVERT INSTALLATION AND CONSTRUCTION CONSIDERATIONS

The invert of the existing culvert at the centerline of highway is at a depth of approximately 7.3 m (elevation 319.5 m). The invert elevations of this culvert between the existing inlet and the outlet are essentially level. Accordingly, a minimum 7.6 m deep excavation (i.e. to elevation 319.2 m) at the centerline of highway will be required in consideration of a 300 mm thick layer of bedding/embedment material.

5.4.1 Staged Excavation

The present platform width at this location is some 13 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 in the east-west (left-right) direction using staged construction unless a temporary shoring is installed generally along the centerline of the highway. A combination of open excavation with temporary shoring installed to a sufficient embedded depth, at least half of excavation height, below the bottom of excavation can be considered for a protection system along the centerline of the highway. Construction using staged sequencing and limiting traffic flow to one lane would be required (see Figure No. SK-2, Appendix 5).

A possible construction plan for a continuous open cut excavation with limited 24/7 traffic control operations, as shown on Figure No. SK-2, Appendix 5, is as follows:

- Stage 1
 1. Limit traffic, with a minimum platform width of 6 m one side (right/west) of the centerline of the highway, under 24/7 traffic control.
 2. Install a temporary protection (shoring) wall along the centerline of highway and beyond culvert to the north and south to the sufficient depths in consideration of the traffic loads, control groundwater and support excavation walls for a total length of about 33 m, and open cut excavate, to the left (east).
 3. Remove embankment to the north and south, slope as required, and remove left (east) section of the existing culvert.
 4. Construct the new culvert to the left (east), about 10 m in length, with adequate backfilling as stated in Section 5.3.1.
 5. Reconstruct the embankment on the left (east), up to a minimum platform width of 6 m for traffic.
- Stage 2

1. Divert the single lane of traffic to the left (east) and continue open excavation to install the remainder of the culvert on the right (west).
2. As the width of the platform increases on the right, the vertical alignment can be raised to final grades, and the traffic can revert back to two lanes when sufficient width permits.

5.4.2 Closed Shoring

An alternative option to consider would be to construct a protection system (shoring walls) around the boundary of the complete excavation area, some 25 m in length and 13 m in width (see Figure No. SK-3, Appendix 5).

A possible staging plan for a protection (temporary shoring wall) system with portable temporary traffic signals, as shown on Figure No. SK-3, Appendix 5, is as follows:

1. Install shoring/protection system around full excavation and along approximately centreline (two celled), to a sufficient depth to consider the traffic loads, control groundwater and support excavation walls.
2. Limit traffic to a single lane on the right (west) with a minimum platform width of 6 m. Undertake excavation to the left (east) to allow construction for the east half of the box culvert.
3. After backfilling the left (east) section to grade, divert the traffic to the left and repeat process on the right (west) side.

5.4.3 Protection System

As noted above, consideration could be given to constructing a vertical wall approximately, along centerline, or constructing a two celled closed shoring system around the boundary of whole construction area for use as a temporary protection system.

Considering the nature of this preliminary foundation investigation, only a single borehole was advanced through the embankment. Additional boreholes through the embankment, up and down chainage from the culvert should be advanced to provide additional information for the design of protection systems.

The installation of a protection system for use in the culvert replacement operation will require penetration through some 7.6 m of granular fills. The embankment fill, at the single borehole location, is generally underlain by some 7 m of native soils consisting of very loose sand and silt underlain by loose to compact silt underlain by the bedrock. As such, a temporary vertical wall for a protection system can likely consist of interlocking sheet piles, or a concrete caisson wall, with sufficient embedded depth below the bottom of excavation. Conceptual shoring locations and profiles are illustrated on Figure Nos. SK-2 and SK-3, Appendix 5.

Considering the cohesionless nature of the embankment fills (granular pavement structure over granular fills) 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 coefficient, as described in Section 5.5,

γ = unit weight, as described in Section 5.5, and

H = height of wall above the base of excavation.

Surcharge loads from the active lane of traffic must also be considered during design of the temporary shoring system.

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

A table outlining the possible temporary excavation protection/flexible retaining systems and their relative advantages, disadvantages, and costs, as well as comments on the viability of the methods is provided in Table A in Appendix 5.

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

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.

5.5 EXCAVATION, DEWATERING, AND EMBANKMENT CONSTRUCTION

All temporary 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 in fill and/or native materials will have to be cut back to an angle of 2H:1V, possibly flatter, dependent upon the Contractors' chosen method of controlling the groundwater.

Excavations must be maintained in a dewatered condition during excavation and foundation construction, and every 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 levels in the creek were measured at elevation some 321.7 m at the inlet area and 321.9 m at the outlet area, respectively, at the time of this investigation. The groundwater levels were measured at elevations 321.7 m, 321.8 m, and 321.8 m at Borehole Nos. 1 to 3, respectively, at the time of this investigation. As such, a head of water some 3 m in depth will have to be controlled during construction. During construction, an effective groundwater control

method, such as a vacuum well point system, etc., should be considered by the contractor to maintain a stable excavation base.

In order to dewater the culvert location, a cofferdam will be required at both the inlet and outlet. The closed shoring system extending to areas of inlet and outlet as stated in Section 5.4.2 would also act as a cofferdam. Alternatively, a temporary gravity type cofferdam is an alternative method of controlling the creek flow. A gravity type cofferdam can be constructed of: earth fill with a low permeable core, sand bag/metre bag, or aquadam (water filled bladder) type dam. Depending upon the base width of the cofferdam seepage may develop below the temporary sand type cofferdam wall. This may require groundwater control by pumping from filtered sump holes within the excavation area or possibly the installation of well points. Sufficient temporary bypass pipes need to be installed through the existing embankment for flow diversion.

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.

5.5.1 Bedrock Excavation

Bedrock was encountered at relatively deep depths (elevations ranging from 316.6 m to 312.4 m) below the existing culvert invert at the locations of three boreholes. As such, it is not anticipated that the bedrock will be encountered within the anticipated depth of excavation.

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	GRANULAR FILL	SAND AND SILT	SILT
Unit Weight (kN/m ³)	22.8	21.2	19	18.5	18
Angle of Internal Friction	34°	31°	30°	26°	28°
Coefficient of Active Earth Pressure (K_a)	0.28	0.32	0.33	0.39	0.36
Coefficient of Passive Earth Pressure (K_p)	3.54	3.12	3.0	2.56	2.77
Coefficient of Earth Pressure at Rest (K_o)	0.44	0.48	0.5	0.56	0.53

For rigid structures, such as a precast concrete culvert, deflection cannot occur, as such the “at-rest” condition (K_o) applies. For flexible structures, such as CSP/HDPE culverts, deflection can occur, as such the “active” condition (K_a) applies.

5.7 CONSTRUCTION CONCERNS

Considering the nature of the granular fill embankment, no unusual construction concerns are anticipated if construction is carried out in general conformance with the above discussion.

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

MACRO KEY PLAN

Drawing No.1

NOT TO SCALE



**FINAL PRELIMINARY
FOUNDATION INVESTIGATION
AND DESIGN REPORT**
GWP 5333-11-00
Highway 60
Station 11+540 Culvert
Township of Sinclair



Reference No: 14/07/14083-F4

April 2015

Appendix 2 Subsurface Data

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

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
WH	Sampler advanced by static weight of hammer and/or rods
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 14/07/14083 DATUM Geodetic LOCATION N 5021920.7 E 342654.6 - Franklin Twp., Station 15+941 ORIGINATED BY JL
 PROJECT GWP 5333-11-00, Highway 60 - F6 BOREHOLE TYPE Truck Mounted CME 45 - Hollow Stem Augers COMPILED BY SH
 CLIENT AECOM DATE (Started) 26 August 2014 TIME
 DATE (Completed) 26 August 2014 (Completed) 5:30:00 PM 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								
322.1	Ground Surface												
0.0	FILL - silty sand, trace gravel Occasional rootlets brown		1	SS	WH								
321.5													
0.6	SAND and SILT, trace clay grey (very loose)		2	SS	WH								0 42 57 1
			3	SS	WH								
			4	SS	2								
319.2													
2.9	SILT, sandy, trace gravel, trace clay grey (loose/very dense)		5	SS	5								6 33 59 2
			6	SS	11								
			7	SS	59								
316.6													
5.5	Auger Refusal Start rock coring BEDROCK - grey gneiss good to excellent quality		8	RC	Rec=100% ROD=91%								
			9	RC	Rec=100% ROD=74%								
313.5													
8.6	End of Sampling End of Borehole												

WATER LEVEL RECORDS	
Date (dd/mm/yy)/Time	Water Depth (m)
1) 26/8/14 5:30:00 PM	0.36
2)	-
3)	-

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

3, X3: Numbers on right refer to Sensitivity
 Numbers on left refer to values greater than 120 kPa
 ○ 3% STRAIN AT FAILURE

MEL-GEO 14083 - BOREHOLE LOGS - F6.GPJ MEL-GEO.GDT 22/4/15

METRIC

RECORD OF BOREHOLE NO. 2



REFERENCE 14/07/14083 DATUM Geodetic LOCATION N 5021941.6 E 342661.9 - Franklin Twp., Station 15+925 ORIGINATED BY JL
 PROJECT GWP 5333-11-00, Highway 60 - F6 BOREHOLE TYPE Track Mounted CME 45 - Hollow Stem Augers COMPILED BY SH
 CLIENT AECOM DATE (Started) 3 September 2014 TIME (Completed) 2:30:00 PM CHECKED BY MAM
 DATE (Completed) 3 September 2014

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100	PLASTIC LIMIT W _p NATURAL MOISTURE CONTENT W LIQUID LIMIT W _L WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA (SI CL)
ELEV DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	"N" VALUES						
326.5	Ground Surface										
0.0	102 mm Asphalt 203 mm Crush Gravel FILL - sand, with to trace gravel, trace silt and clay brown compact		1	SS	21		326		○		24 69 (7)
			2	SS	15		325		○		
			3	SS	16		324		○		
			4	SS	9		323		○		
			5	SS	11		322		○		
			6	SS	14		321		○		
322.1							320		○		
4.4	SAND and SILT, trace gravel, trace clay trace rootlets and wood pieces grey (compact/very loose)		7	SS	14		319		○		9 45 (46)
			8	SS	4		318		○		
			9	SS	3		317		○		0 53 45 2
			10	SS	4		316		○		
			11	SS	4		315		○		
			12	SS	4		314		○		
316.4											
10.1	SILT, sandy to trace sand, trace gravel, trace clay grey (loose/compact)		13	SS	WH				○		
	Continued Next Page										

COMMENTS

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

+ 3, × 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) 3/9/14 2:30:00 PM	4.9	▽ - 1.5
2) 4/9/14 1:30:00 PM	4.67	▽ -
3)	-	▽ -

MEL-GEO 14083 - BOREHOLE LOGS - F6.GPJ MEL-GEO GDT 22/4/15

METRIC**RECORD OF BOREHOLE NO. 2**

REFERENCE 14/07/14083 DATUM Geodetic LOCATION N 5021941.6 E 342661.9 - Franklin Twp., Station 15+925 ORIGINATED BY JL
 PROJECT GWP 5333-11-00, Highway 60 - F6 BOREHOLE TYPE Track Mounted CME 45 - Hollow Stem Augers COMPILED BY SH
 CLIENT AECOM DATE (Started) 3 September 2014 TIME
 DATE (Completed) 3 September 2014 (Completed) 2:30:00 PM CHECKED BY MAM

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA (SI CL)
ELEV DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100		
	Continued from Previous Page													
311.6			14	SS	14									4 7 82 7
14.9	Auger Refusal Start rock coring BEDROCK - grey to pink gneiss good quality		15	RC	Rec= 100% RQD= 79%									
			16	RC	Rec= 100% RQD= 89%									
308.5														
18.0	End of Sampling End of Borehole													

MEL-GEO 14083 - BOREHOLE LOGS - F6.GPJ MEL-GEO.GDT 22/4/15

METRIC

RECORD OF BOREHOLE NO. 3



REFERENCE 14/07/14083 DATUM Geodetic LOCATION N 5021952.3 E 342685.8 - Franklin Twp., Station 15+925 ORIGINATED BY JL
 PROJECT GWP 5333-11-00, Highway 60 - F6 BOREHOLE TYPE Truck Mounted CME 45 - Hollow Stem Augers COMPILED BY SH
 CLIENT AECOM DATE (Started) 27 August 2014 TIME
 DATE (Completed) 27 August 2014 (Completed) 5:00:00 PM 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								
323.4	Ground Surface												
0.0	SAND and SILT, trace clay brown to grey (very loose/loose)		1	SS	5		323						
			2	SS	2		322						
			3	SS	1		321						
			4	SS	3		320						
			5	SS	WH		319						
			6	SS	3		318						
			7	SS	3		317						
			8	SS	WH		316						
			9	SS	WH		315						
			10	SS	5		314						
313.3	SILT, sandy, trace clay grey (very dense)		11	SS	75/51mm		313						
312.4	Auger Refusal Start rock coring						312						
11.0	BEDROCK - grey gneiss good quality		12	RC	Rec=98% RQD=79%		311						
Continued Next Page													
COMMENTS								+ 3, × 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) 27/8/14 5:00:00 PM		1.55		2.24	
								2)		-		-	
								3)		-		-	
The stratification lines represent approximate boundaries. The transition may be gradual.													

MEL-GEO 14083 - BOREHOLE LOGS - F6.GPJ MEL-GEO GDT 22/4/15

METRIC**RECORD OF BOREHOLE NO. 3**

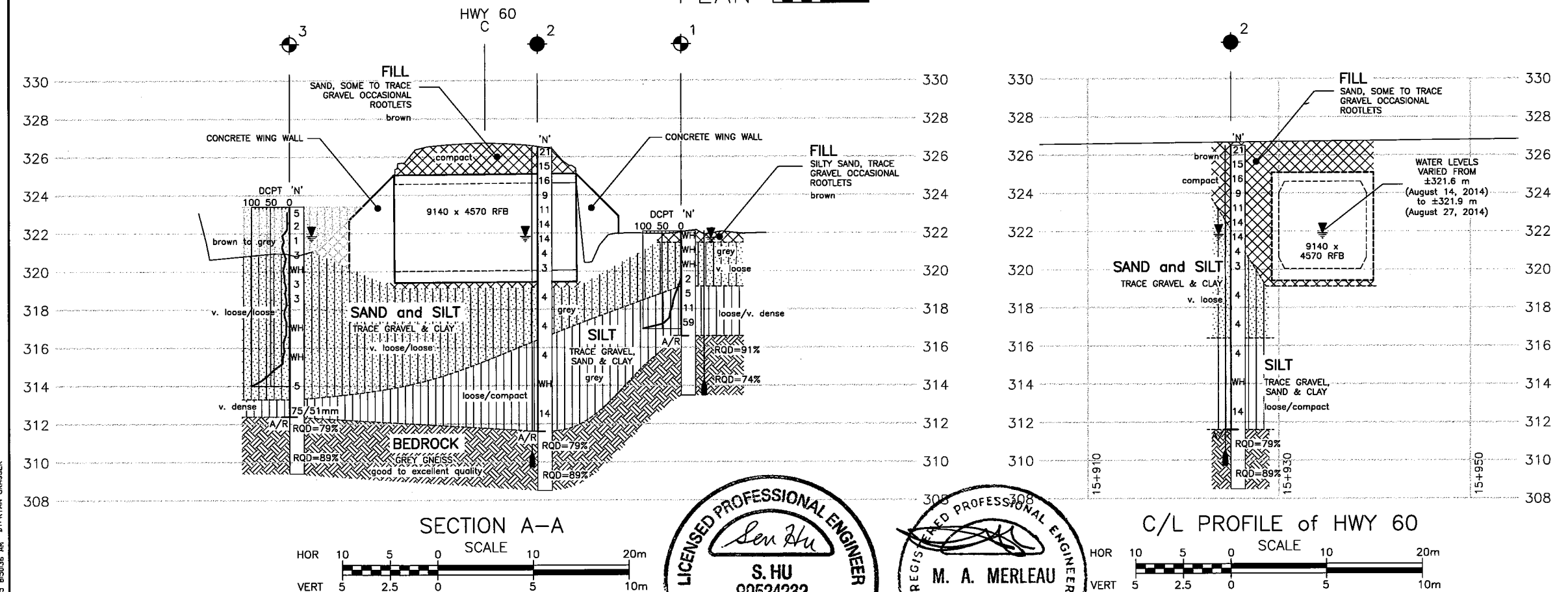
REFERENCE 14/07/14083 DATUM Geodetic LOCATION N 5021952.3 E 342685.8 - Franklin Twp., Station 15+925 ORIGINATED BY JL
 PROJECT GWP 5333-11-00, Highway 60 - F6 BOREHOLE TYPE Truck Mounted CME 45 - Hollow Stem Augers COMPILED BY SH
 CLIENT AECOM DATE (Started) 27 August 2014 TIME
 DATE (Completed) 27 August 2014 (Completed) 5:00:00 PM 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					
	Continued from Previous Page		13	RC	Rec=100% RQD=84%		310										
309.4 14.0	End of Sampling End of Borehole																

MEL-GEO 14083 - BOREHOLE LOGS - F6.GPJ MEL-GEO.GDT 22/4/15

Appendix 3 Borehole Plan and Lab Data

Drawing No. 2: Borehole Location and Soil Strata
Figure Nos. L-1 to L-3: Grain Size Distribution Curves
Table No. L-4: Laboratory Test Summary Sheet



A circular professional engineer seal for the Province of Ontario. The outer ring contains the text "LICENSED PROFESSIONAL ENGINEER" at the top and "PROVINCE OF ONTARIO" at the bottom. Inside the ring, the name "S. HU" is written in a stylized script. Below the name is the license number "90524232". At the bottom of the seal, the expiration date "15/04/24" is written in a stylized script.

308 308

REGISTERED PROFESSIONAL ENGINEER

M. A. MERLEAU

15/04/24

PROVINCE OF ONTARIO

DRAWING NOT TO BE SCALED
50mm ON ORIGINAL DRAWING

DISTRICT CULT. No. GWP No. 5333-11-00	
HWY 60 CULVERT AT STATION 15+935 FRANKLIN TOWNSHIP	DRAWING 2
BOREHOLE LOCATIONS AND SOIL STRATA	2
<div style="border: 2px solid black; padding: 5px; display: inline-block;"> LVM Merlex </div>	METRIC

KEY PLAN
 N.T.S.

LEGEND	
	Borehole
	Borehole w/ Dynamic Cone Penetration Test
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	O/S	NORTHING	EASTING
1	322.1	20.5m Rt	5021920.7	342654.6
2	326.5	5.5m Lt	5021941.6	342661.9
3	323.4	20.5m Lt	5021952.3	342685.8

NOTES:
 1. 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.
 2. Base plan and alignment provided in digital format by exp. on November 24, 2014.
 3. The size of opening of the culvert was established from Contract Drawing titled "General Plan" (No. D-4910-1) dated October, 1961 (W.P. 208-60)

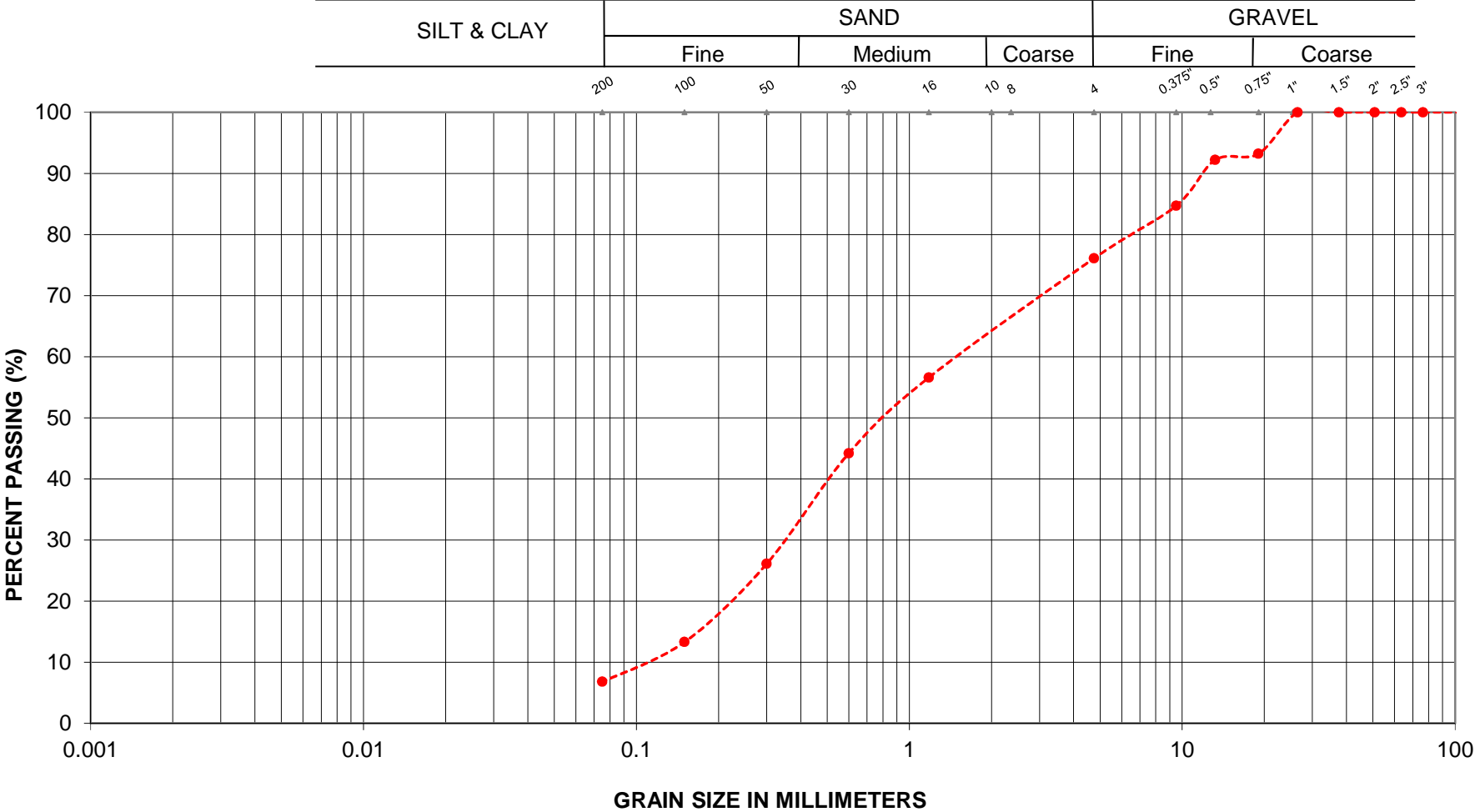
GEOCRETS No. 31E-348

15	RG	DRAFT							
15	RG	FINAL							

DESCRIPTION									
CHK	SH	CODE		LOAD		DATE	MAR/15		
RG	CHK	SH	SITE	STRUCT	SCHEME	DWG	2		



GRAIN SIZE ANALYSIS



---●--- BH No.: 2 Sa No.: 2 Depth: 0.76 - 1.22 m

SAND FILL

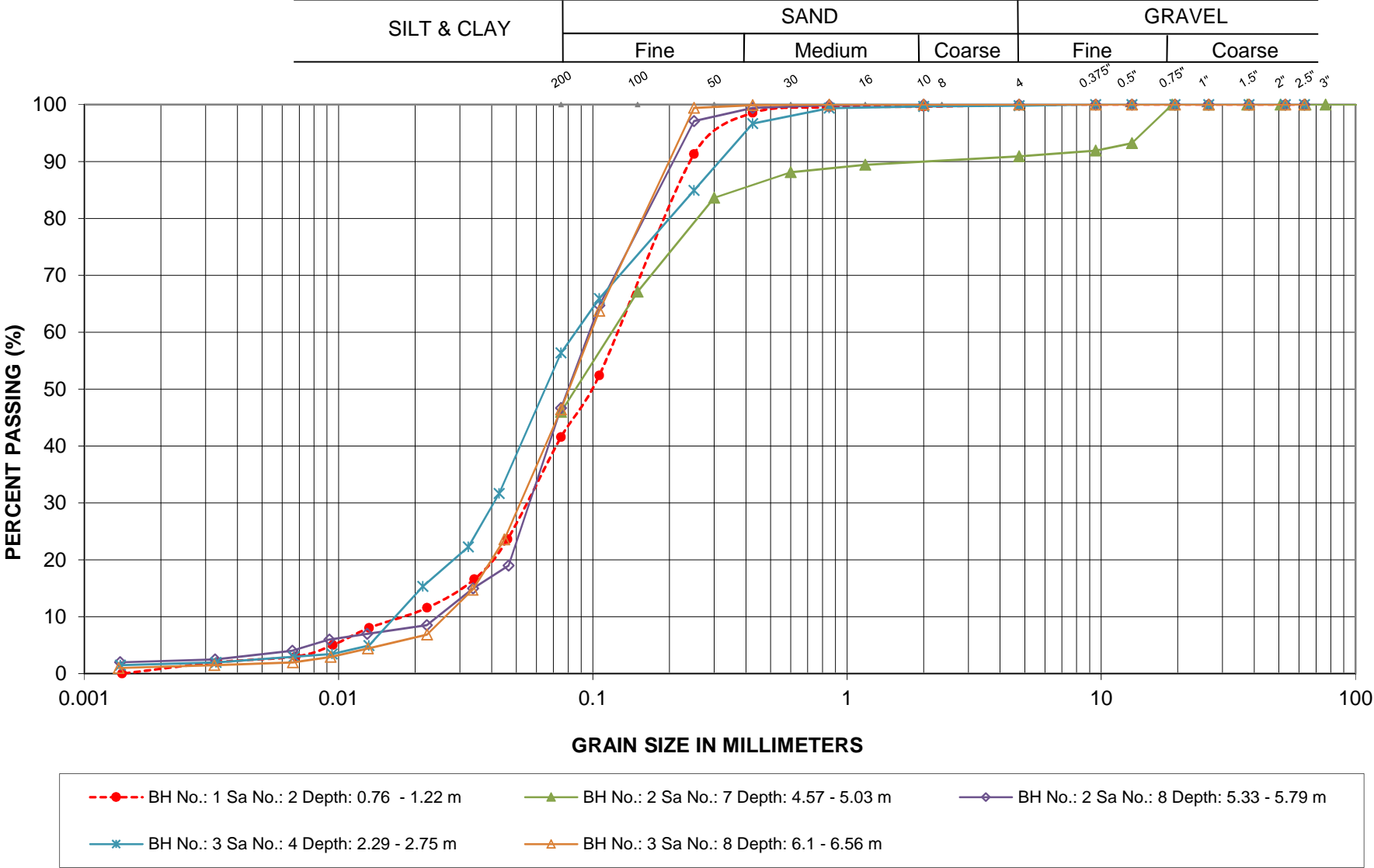
LOCATION: Hwy 60 Sta. 15+935 Culvert
TWP. Franklin, Ontario

LVM-Merlex, a Division EnGlobe Corp.

FIGURE L-1



GRAIN SIZE ANALYSIS



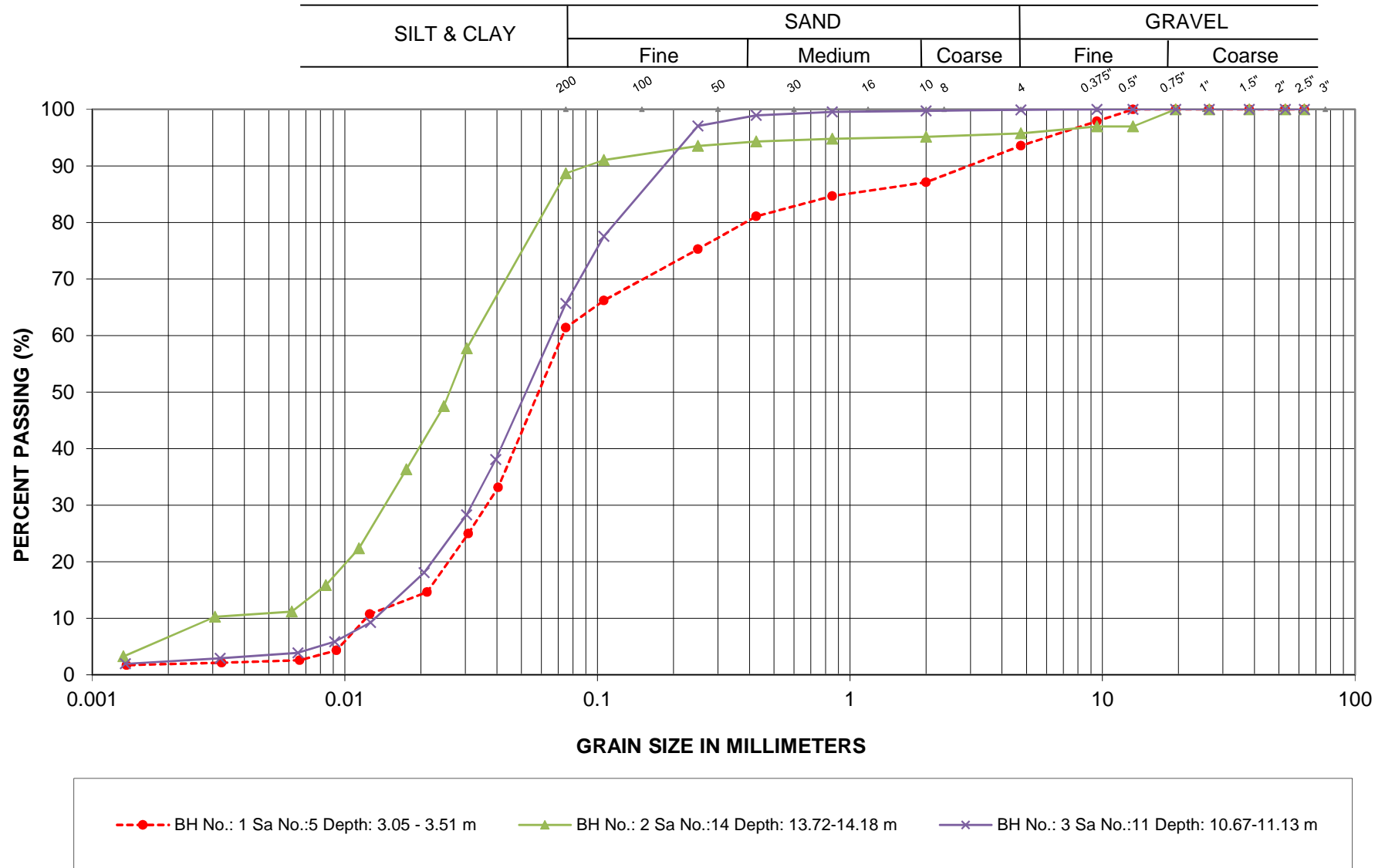
SAND AND SILT

LOCATION: Hwy 60 Sta. 15+935 Culvert
TWP. Franklin, Ontario

LVM-Merlex, a Division EnGlobe Corp.

FIGURE L-2

GRAIN SIZE ANALYSIS



Sandy SILT to SILT

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	0.3					58.0				WH			
	2	1.0	0	42	57	1	52.1				WH			
	3	1.8					33.2				WH			
	4	2.5					48.5				2			
	5	3.3	6	33	59	2	26.6				5			
	6	4.0					28.1				11			
	7	4.8					25.2				59			
2	1	0.2					4.2				21			
	2	1.0	24	69	7		3.5				15			
	3	1.8					4.8				16			
	4	2.5					6.0				9			
	5	3.3					6.9				11			
	6	4.0					17.7				14			
	7	4.8	9	45	46		16.5				14			
	8	5.6	0	53	45	2	46.6				4			
	9	6.3					29.3				3			
	10	7.9					26.4				4			
	11	9.4					25.0				4			
	12	10.9					26.9				4			
	13	12.4					27.3				WH			
	14	14.0	4	7	82	7	28.5				14			
3	1	0.3					13.8				5			
	2	1.0					17.8				2			
	3	1.8					32.5				1			
	4	2.5	0	44	54	2	28.2				3			

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 (%)				
3	5	3.3					23.1				WH			
	6	4.0					29.7				3			
	7	4.8					26.6				3			
	8	6.3	0	54	45	1	26.3				WH			
	9	7.9					25.9				WH			
	10	9.4					18.8				5			
	11	10.9	0	34	64	2	21.1				75/51mm			

Appendix 4 Photo Essay

Enclosure No. 5:

Photo Essay

Embankment at Culvert Location – Looking North

Photo: 1



Culvert Inlet – Looking North

Photo: 2



Project: Hwy 60 – Culvert 15+935

Photos Provided By: LVM

Date: August 2014

Creek Downstream at Culvert Inlet– Looking West (Right Side)

Photo: 3



Creek Upstream at Culvert Outlet – Looking East (Left Side)

Photo: 4



Project: Hwy 60 – Culvert 15+935

Photos Provided By: LVM

Date: August 2014

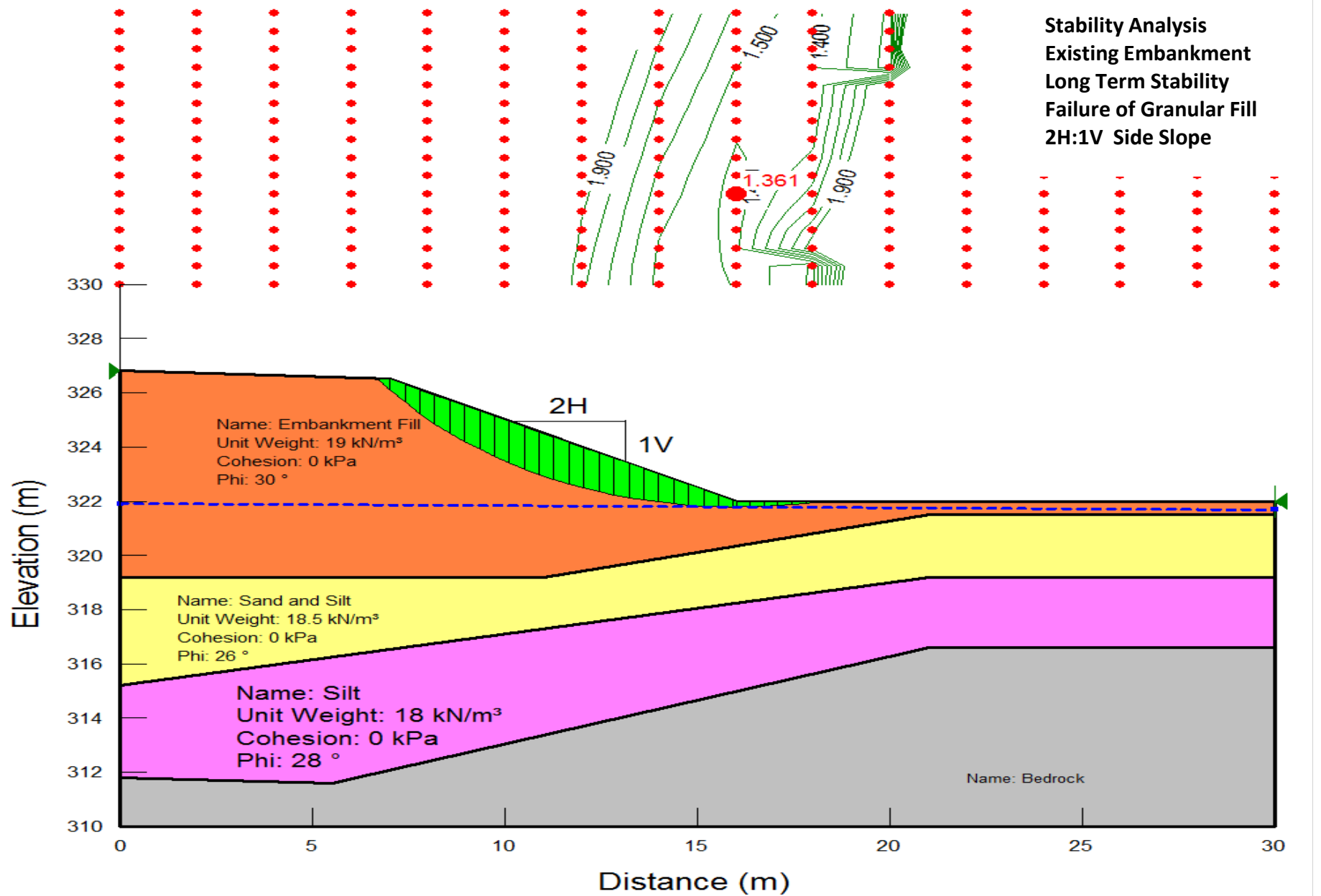
Appendix 5 Design Data

Figure No. S-1: Slope Stability Analyses

Table A: Comparison of Shoring Alternatives

Figure No. SK-2: Conceptual Staged Excavation Plan/Profile

Figure No. SK-3: Conceptual Closed Shoring Plan/Profile



Station 15+935

Township of Franklin

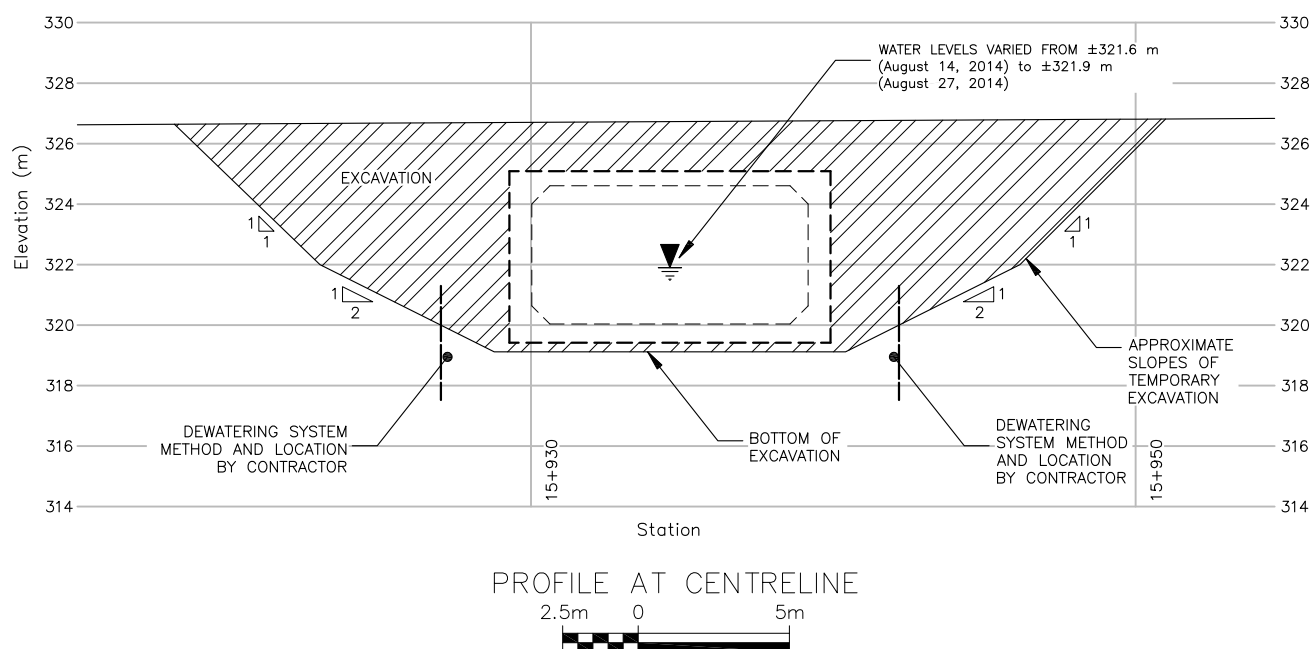
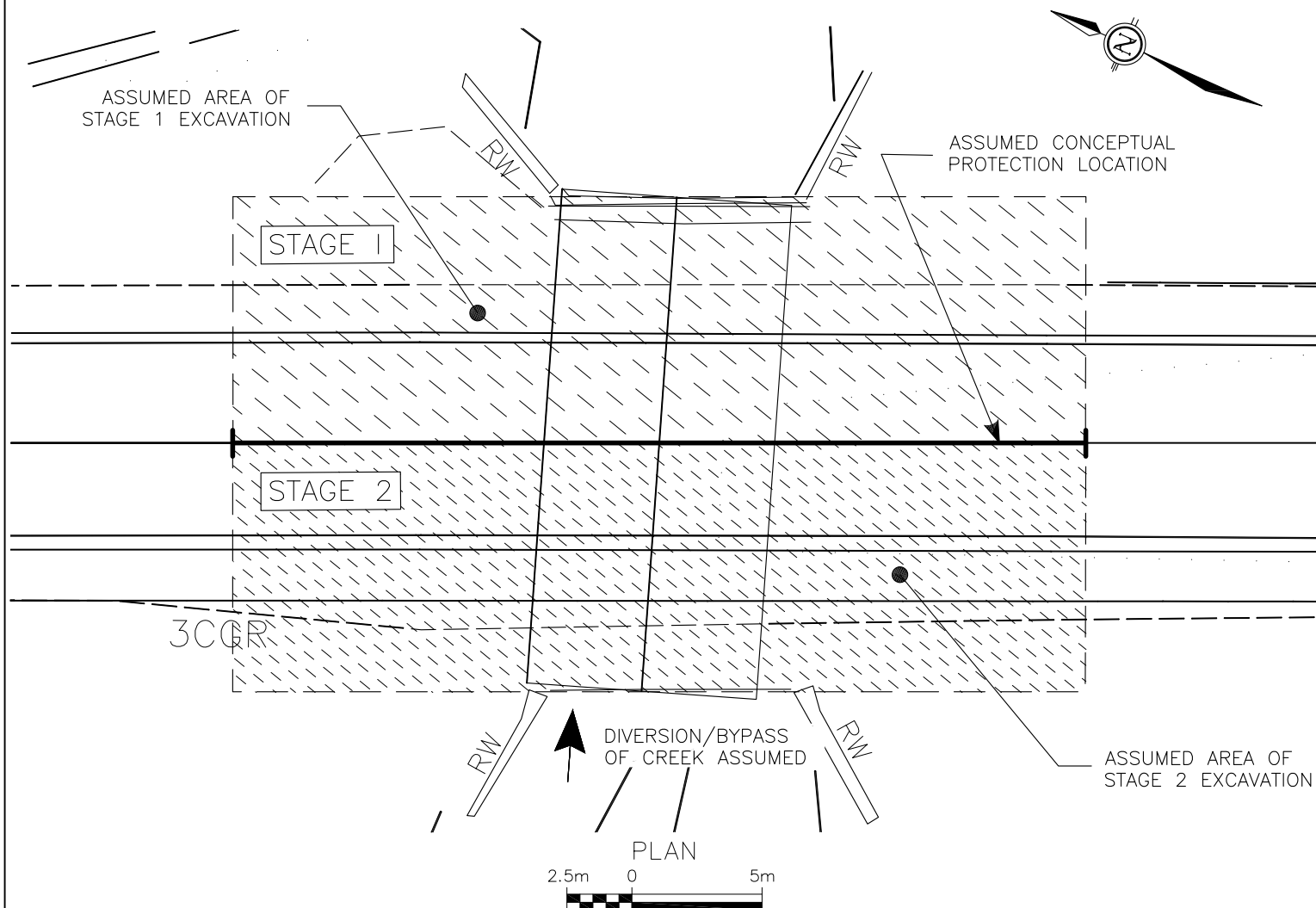
Project: G.W.P 5333-11-00

Location: Hwy 60, Boyne Creek Culvert

Figure No. S-1

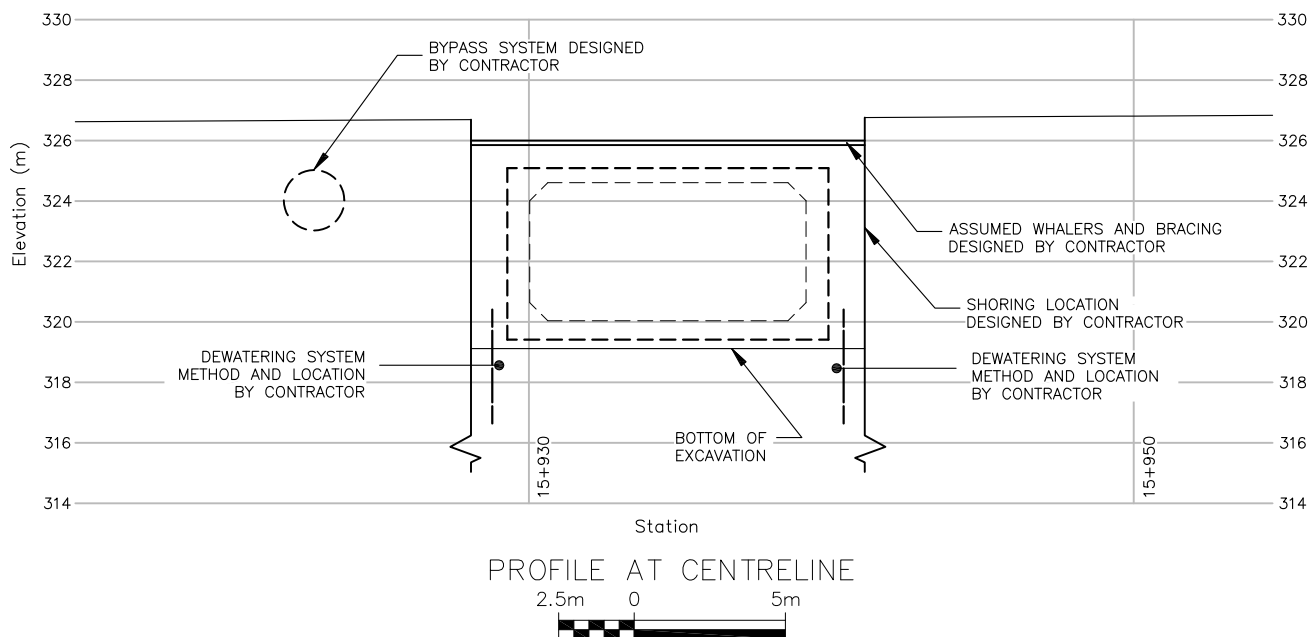
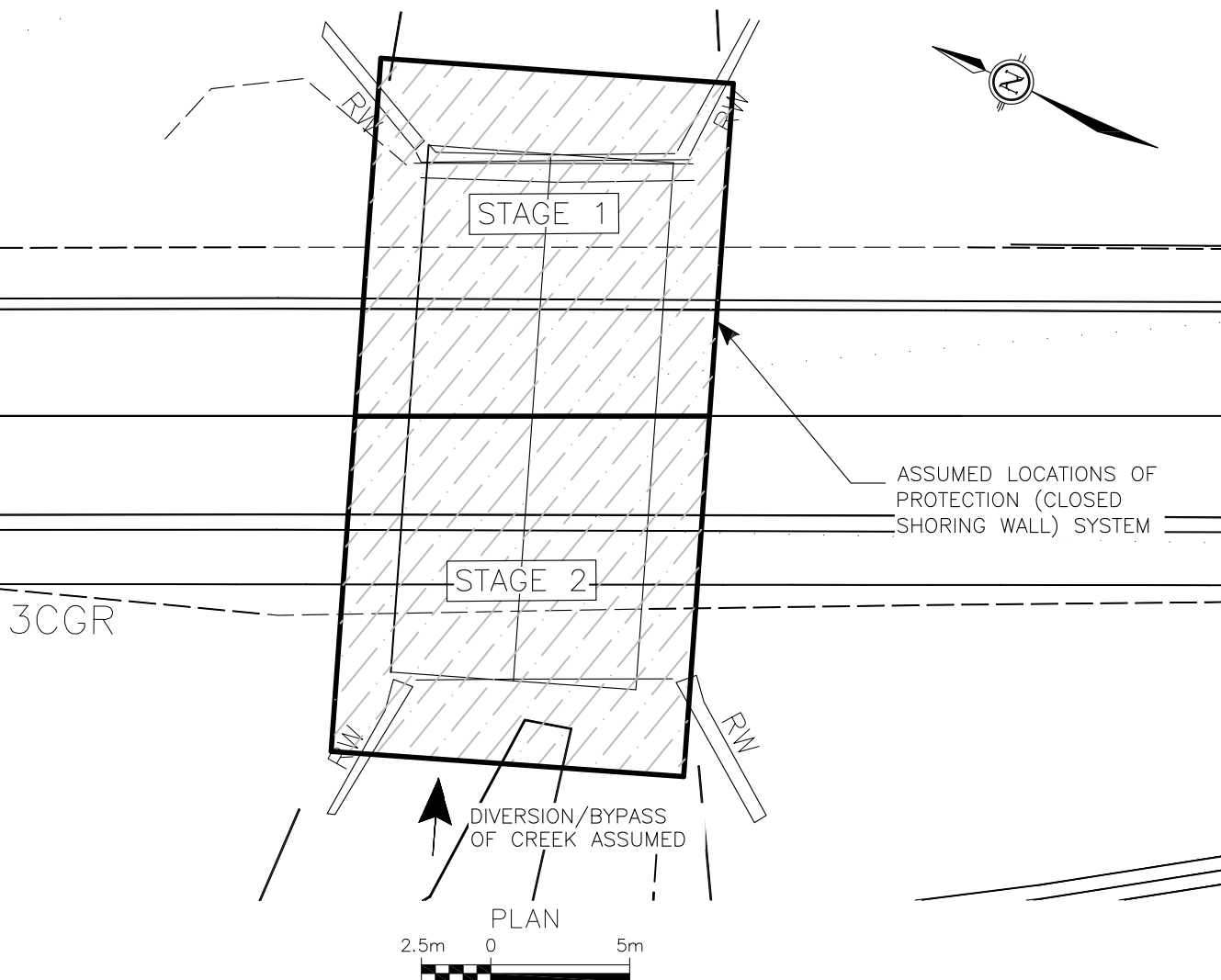
Table A – Comparison of Shoring Alternatives

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 recommended due to high groundwater and permeable granular fills	\$ 650/m ²
Steel Sheet Piles	5 – 21	-High strength, continuous, -Readily available	-Limited by soil conditions (i.e. obstructions)	Recommended, breaking the concrete box in advance and temporary support inside box culvert required	\$ 650/m ²
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)	Feasible, but higher cost	
Soldier piles	5 – 25	-Easy installation -Readily available -Adaptable to various ground conditions	-Pre-drilling may be required -Possible ground loss	Not considered due to high groundwater level and permeable granular fills	\$ 725/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	Feasible	
Concrete Diaphragm	10 – 30	-High Strength -Durable -Can be permanent	-High cost -Requires specialized equipment/control	Not Considered due to higher costs	
Micropiles with reinforced shotcrete face		-Can be installed in various ground conditions -High strength -Good tolerance	-High Cost -Requires specialized equipment	Feasible with adequate dewatering	\$ 900/m ²



Highway 60, Township of Franklin - Culvert at Station 15+935
Conceptual Staged Excavation Plan & Profile

FIGURE SK-2



Highway 60, Township of Franklin - Culvert at Station 15+935
Conceptual Closed Shoring Plan & Profile

FIGURE SK-3

