

**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
Highway 60
Station 13+029 – Township of Franklin
GWP 5333-11-00**

FINAL PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT

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Final Preliminary Foundation Investigation and Design Report

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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. The site is located at Station 13+029 in the Township of Franklin on Highway 60, some 400 m north of the intersection between Highway 60 and South Portage Road of MR 9 (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 Corrugated Steel Pipe (CSP) culvert is located on Highway 60 at Station 13+029 in the Township of Franklin. The topography of this site is located in a valley area. The existing highway embankment currently supports two undivided lanes of highway. At the culvert location the highway runs locally in a north-south direction. A north-south orientation is used in this report for description purpose. The existing highway, at the culvert location, is constructed on a granular fill mixed with rockfill embankment some 4 m to 7.5 m in height, with centerline elevation of 367.4 m at the culvert location. The existing embankment slopes, in the area of the culvert, have been built between slope angles of approximately 1.5H:1V on the western (right) side to 1.7H:1V to 2H:1V on the eastern (left) side. The culvert at this location is a 910 mm diameter Corrugated Steel Pipe (CSP) culvert, some 34 m in length. The flow through the culvert is from the west to the east (right to left).

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 sands 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 25th and September 18th, 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 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 selected 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-4 and Table No. L-5).

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 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 13+029, 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 exploration program, three (3) sampled boreholes were put down at this site, with Borehole No. 1 advanced at the culvert inlet, Borehole No. 2 advanced through the embankment, and Borehole No. 3 advanced at the culvert outlet. At the time of the subsurface investigation, the ground surface elevations at Boreholes Nos. 1 to 3 were recorded at elevations 365.1 m, 367.2 m, and 359.9 m, respectively.

4.1.1 Pavement Structure

Borehole No. 2 was advanced through the embankment where a pavement structure consisting of 50 mm asphalt and 250 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 fill consisting of brown sand some to with gravel, mixed with rock fill, trace to some silt trace clay was penetrated. The natural moisture content measured on samples of this deposit was in the order of 3% to 25%. Gradation analyses were carried out on two (2) samples of this deposit, the results of which indicated 17% to 23% gravel size particles, 64% to 72% sand size particles, and 5% to 19% silt and clay size particles (Figure No. L-1, Appendix 3). Boulders/rockfill were encountered at depths from 1.5 m to 2.3 below ground surface which resulted in sampler refusal during the 3rd SPT test at Borehole No.2. Based on SPT 'N' values of 4 to 38 blows per 300 mm penetration and refusal, the compactness of this deposit was described as loose to very dense. This deposit was encountered to depths of 0.7 m and 3.7 m below ground surface at Borehole Nos. 1 and 2 (elevations 364.4 m and 363.5 m, respectively).

4.1.3 Organic Soils

At ground surface at Borehole No. 3, a layer of organic silty sand was penetrated. The natural moisture content measured on one (1) sample of this layer was some 109%. This organic soil layer was encountered to an approximate depth of 0.7 m below ground surface at Borehole No. 3 (elevation 359.2 m).

4.1.4 Sand (Gravelly to Some Gravel)

Underlying the sand fill at Borehole No. 1 and the organic soil at Borehole No.3, a layer of dark brown gravel and sand to grey gravelly sand with silt was penetrated. Underlying the granular fill at Borehole No.2 and the gravel and sand deposit at Borehole No. 1, a layer of grey sand

trace to some silt, some silt and clay was penetrated. Cobble and boulder size rock pieces were encountered in this deposit at Borehole No. 1.

The natural moisture content measured on samples of this deposit was in the order of 11% to 30%. The gradation analysis was carried out on three (3) samples of this deposit, the results of which indicated 15% to 52% gravel size particles, 42% to 69% sand size particles, and 4% to 23% silt and clay size particles (Figure Nos. L-2 to L-4 in Appendix 3). Based on SPT 'N' values of 26 to 45 blows per 300 mm penetration to 25 blows per 51 mm penetration, this deposit was described as compact to very dense. This deposit was encountered to depths of 3.1 m, 5.6 m, and 2.1 m below grade at Borehole Nos. 1 to 3, respectively (elevations 362.0 m, 361.6 m and 357.8 m, respectively).

4.1.5 **Bedrock**

Underlying the above described gravelly sands at Borehole No. 3 and sands at Borehole Nos. 1 and 2, the bedrock was proven by diamond core drilling. The bedrock was described as greyish brown to grey gneiss bedrock. Based on Rock Quality Designation (RQD) values of 35% to 89%, the bedrock was described as poor to good quality. Sampling in the bedrock was terminated at depths of 6.1 m, 8.7 m, and 5.2 m below grade at Borehole Nos. 1 to 3, respectively (elevations 359 m, 358.5 m, and 354.7 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**

At the time of this investigation (August 25th, 2014), the creek water level was measured at elevation some 359.6 m at the outlet area.

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 No. 3 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 363.8 m, 364.9 m, and 358.9 m at Borehole Nos. 1 to 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 a CSP culvert as identified by the MTO.

The existing culvert, located at Station 13+029, in the Township of Franklin, is a 910 mm diameter CSP culvert some 34 m long. The invert of the existing culvert at centerline is at a depth of some 5.8 m (elevation 361.8 m). 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 structure at this site has been constructed using a granular pavement structure overlying granular fills mixed with rock fill. The native material, underlying the embankment fill, generally consisted of dense sands overlying the bedrock at a relatively shallow depth.

The type of culvert (concrete, CSP, or High Density Polyethylene (HDPE)) to replace the existing culverts is currently unknown. However, it is understood that the new culvert will be constructed along a similar skew and alignment. It is further understood that the final vertical alignment of the highway will remain essentially the same.

5.2 FOUNDATION CONSIDERATIONS

The founding native dense sands overlying the bedrock present below the existing embankment are considered very good 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 excessively disturbed during construction and groundwater is controlled throughout construction, as discussed in Section 5.6.

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 600 kPa can be used for a closed culvert (i.e. precast concrete frame box culvert or CSP culvert). In consideration of the width of the culvert, depth of overburden, 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, or pipe arch culverts on footings) are considered, then a factored bearing resistance at ULS of 450 kPa, and a geotechnical reaction at SLS of 250 kPa would apply for design, in consideration of 25 mm settlement and taking into consideration the limited depth of overburden and smaller footing width.

5.2.1 Slope Stability

The maximum height of the embankment above the stream bed at this location is some 2 m on the western side slope and 7 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	GRAVEL AND SAND	GRAVELLY SAND
Unit Weight (kN/m ³)	20	20	20
Effective Friction Angle (degrees)	32	34	34

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 1.2 for the existing and in the order of 1.3 for the new embankments except a lower value in the order of 1.1 against the minor surficial slippage on the existing embankment (see Figure Nos. S-1 and S-2, 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 consisted of dense sands to gravel and sand. A review of the condition of the pavement surface, at the culvert locations, 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 Culvert

Concrete pipes can be considered for culvert replacement at this site. A Class B Bedding for the concrete pipes shall consist of Granular A with a thickness of 300 mm. 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 operations. During backfilling, the loose backfill should be placed in uniform lifts not greater than 200 mm in thickness per OPSS 422 at a balanced manner on each side of the pipe. The elevation difference of the backfill on either side of the pipe must be limited to a

maximum 200 mm. Cover material for concrete pipes can consist of Granular A and placed to the dimensions as shown on OPSD 802.031. If circular concrete pipes are used, compaction of the haunch is critical and should be constructed in accordance with OPSS 501.

A precast concrete rigid frame box culvert can also be considered for culvert replacement at this site. However, considering the size of the existing culvert a box culvert would likely not be used. 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 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 300 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 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, it is recommended that clay seals be used at this culvert location.

5.3.2 Flexible Culvert

Flexible culverts (i.e. CSP/HDPE) can also be considered for culvert replacement at this site. If flexible pipes are used for replacement, embedment material should consist of Granular B Type I per OPS.PROV 1010 provided the maximum size of stone inclusions is limited to 25 mm or less in size and placed in accordance with OPSD 802.010 for a Type 3 soil. The material in the haunch area must be compacted to 100% Standard Proctor Dry Density prior to placing the remainder of the embedment material. During backfilling, the loose embedment fill should be placed in uniform lifts not greater than 200 mm in thickness on either side of the culvert per OPSS 401 and at a balanced manner on the outer sides of the culvert units. The elevation difference of the backfill on either side of the culvert must be limited to a maximum 200 mm.

Considering the porous nature of the embankment fill, inlet clay seals along the culvert or outlet cut-off walls are not required; however, 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.

5.4 CULVERT INSTALLATION AND CONSTRUCTION CONSIDERATIONS

The invert depths of the existing culvert range from some 3.5 m below the western edge of embankment (elevation 363 m) at the inlet to some 7.6 m below the eastern edge of embankment (elevation 359.6 m) at the outlet. The invert of the existing culvert at centerline is at a depth of some 5.8 m (elevation 361.8 m). Accordingly a minimum 3.8 m deep excavation (i.e. to elevation 362.7 m) at the western slope and 7.9 m deep excavation (i.e. to elevation 359.6 m) at the eastern slope will be required in consideration of a 300 mm thick layer of bedding/embedment material.

5.4.1 Staged Construction

The present platform width at this location is some 15 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 sliver widening is undertaken. In general, an open cut excavation can be considered if the platform is temporarily lowered by a minimum of some 2 m. If this lowering cannot be accommodated then consideration can be given to a combination of lowering and widening or to constructing a vertical wall for use as a protection system. Additional geotechnical investigation through the embankment, up and down chainage from the culvert, should be compiled to provide additional information for widening the platform, if required.

As noted, the platform at this location, as is, is of insufficient width to carry out an open excavation using staged construction, unless temporarily lowering the vertical alignment 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 (see Figure No. SK-3, Appendix 5).

A possible staging plan for a continuous open cut excavation under traffic control operation, as shown on Figure No. SK-3, Appendix 5, is as follows:

- Locally lower the grade at the culvert to an elevation of approximately 365.6 m.
- Limit traffic to a single lane on the right (west), with a minimum platform width of 6 m, under traffic control with temporary portable traffic lights.
- Open cut excavate, to the left (east), and install approximately 12 m in length of new culvert.
- Reconstruct the embankment on the left (east), with a minimum platform width of 6 m for traffic.

- Divert the single lane of traffic to the left and continue open excavation to install the remainder of the culvert on the right.
- As the width of the platform increases on the right, the vertical alignment can be raised, and the traffic can revert back to two lanes when sufficient width permits.

5.4.2 Protection System

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

Considering the preliminary nature of this 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 protection systems, if required.

The installation of a protection system for use in the culvert replacement operation will require penetration through some 6 m of granular fills mixed with rock fill in which cobbles and boulders were encountered. The embankment fill is generally underlain by some 1 m thick dense sands underlain by the bedrock. As noted rock fill mixed granular fill was encountered at depths from some 1.5 m to 2.3 m below ground surface in the embankment at the borehole location. As such, a temporary vertical wall for a protection system can likely consist of concrete caissons or micropiles with at least 0.5 m embedded depth in the bedrock. Conceptual shoring location and section are illustrated on Figure Nos. SK-4 and SK-5, Appendix 5.

Considering the cohesionless nature of the embankment fills (granular pavement structure over granular mixed with rock 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 an/or native materials will have to be cut back to an angle of 2H:1V, possibly shallower, dependent upon the Contractors' chosen method of controlling the groundwater.

It should be noted that the existing fills and native soils contained varying concentrations of cobble and boulder size rockfill and were in a generally dense state of compactness. As such, the contractor must be prepared to use equipment of sufficient capacity to excavate in this type of material.

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 some 359.8 m at the outlet area at the time of this investigation and the water level in Borehole No. 3 (adjacent to the stream) had stabilized at elevation 358.9 m at the time of this investigation. All excavations extending below the groundwater table, present at the time of construction, will have to be maintained in a dewatered condition. 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. The effectiveness of this method of groundwater control would be limited to conditions where the prevailing groundwater table is less than some 1 m above the final excavation depth. If the excavation must penetrate to a greater depth below the prevailing groundwater table a more effective groundwater control method, such as a vacuum well point system, should be considered by the contractor to maintain a stable excavation base.

A sand bag cofferdam or aquadam could be considered for controlling stream flow, depending upon anticipated flow at time of construction. Considering the dense native soils contained cobbles and boulders, a sheet pile coffer dam is not considered feasible for this site, since adequate penetration of the sheets would be restricted by the presence of boulders and shallow bedrock. The presence of cobbles and boulders could also adversely impact the installation of a well point system. As noted previously, it is probable that if filtered well points are considered

they may have to be installed using a duplex rotary percussive drill to penetrate the native course soils to sufficient depth.

By-pass pumping through a temporary culvert could be carried out to divert the stream flow at the time of construction; however this would require a large pumping capacity, with back-ups. As such, the approach of using the existing culvert as a bypass and constructing a separate new culvert may be a more appropriate approach.

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 a relatively shallow depth below the existing culvert invert. Depending on the invert and type of new culvert, the bedrock may be encountered within the anticipated depth of excavation; therefore bedrock excavation and/or blasting operations may be required.

If blasting is required reference shall be made to OPSS 120. A blast design is required to be provided by the blasting contractor before blasting operations are carried out. A pre-blast survey (OPSS 120.07.03) and/or blast monitoring is probably not required considering the minimal depth and quantity of rock removal and somewhat isolated nature of the site.

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	GRAVEL AND SAND	GRAVELLY SAND
Unit Weight (kN/m ³)	22.8	21.2	20	20	20
Angle of Internal Friction	34°	31°	32°	34°	34°
Coefficient of Active Earth Pressure (K_a)	0.28	0.32	0.31	0.28	0.28
Coefficient of Passive Earth Pressure (K_p)	3.54	3.12	3.25	3.57	3.57
Coefficient of Earth Pressure at Rest (K_o)	0.44	0.48	0.47	0.56	0.56

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 major construction concerns are anticipated if construction is carried out in general conformance with the above discussion. The presence of rockfill in the embankment, mixed with granular soils, could increase the difficulty of installing a shoring wall, especially for the sheet pile. Therefore, estimates of the location and concentration of rock fill pieces in the embankment must be acquired when further foundation investigation is carried out for the detailed design. At this preliminary stage a Notice to Contractor for the request would be appropriate.

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

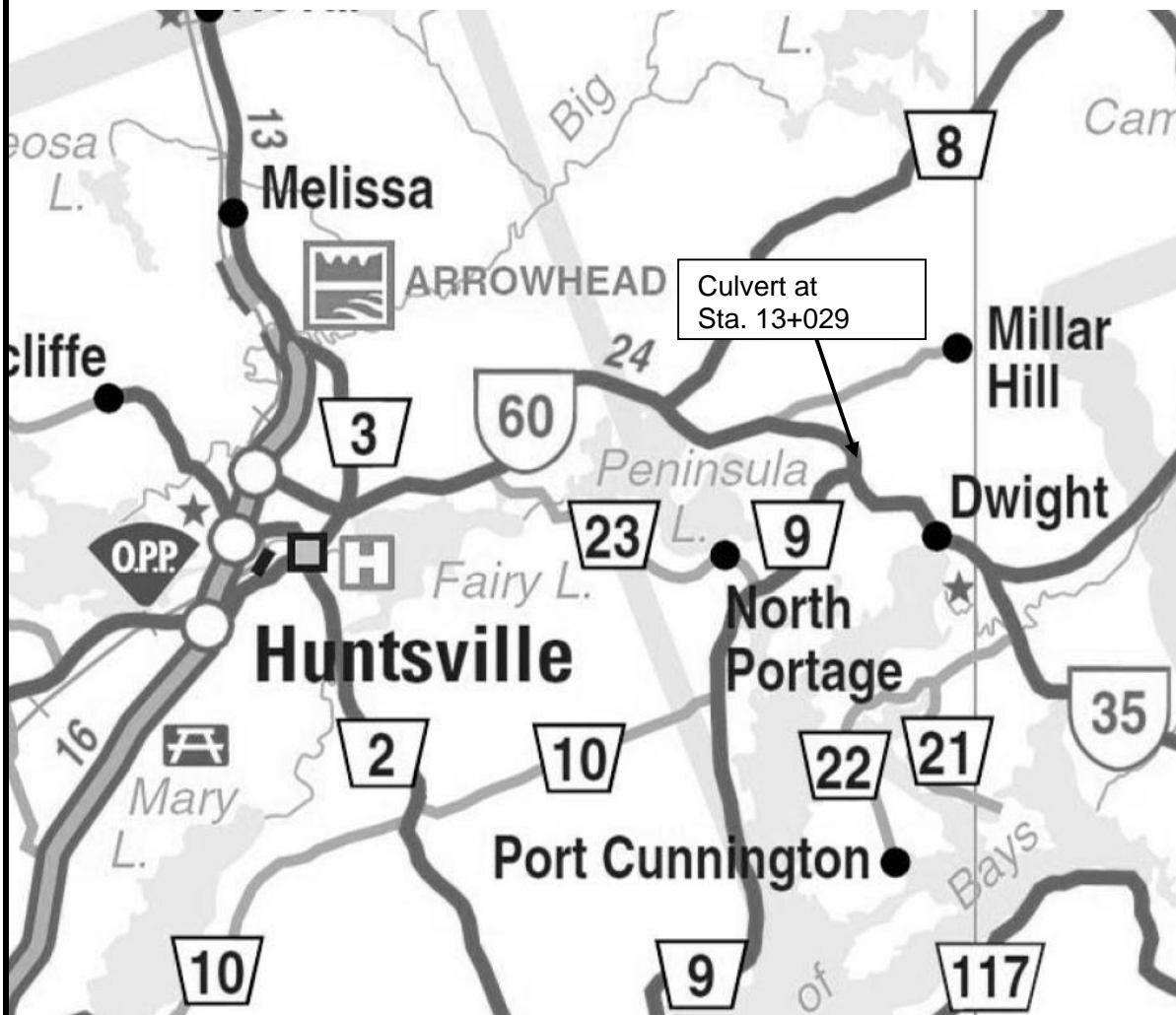
Drawing No. 1

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 13+029 Culvert
Township of Franklin



Reference No: 14/07/13083-F5

March 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.

REFERENCE	14/07/14083	DATUM	Geodetic	LOCATION	N 5023422 E 340863.3 - Franklin Twp., Station 13+033	ORIGINATED BY	JL
PROJECT	GWP 5333-11-00, Highway 60 - F5			BOREHOLE TYPE	Track Mounted CME 45 - Hollow Stem Augers	COMPILED BY	SH
CLIENT	AECOM	DATE (Started)	25 August 2014	TIME (Completed)	5:10:00 PM	CHECKED BY	MAM

MEL-GEO 14083 - BOREHOL LOGS - F5.GPJ MEL-GEO.GDT 27/3/15

METRIC

RECORD OF BOREHOLE NO. 2



REFERENCE 14/07/14083 DATUM Geodetic LOCATION N 5023418.2 E 340871 - Franklin Twp., Station 13+037 ORIGINATED BY JL

PROJECT GWP 5333-11-00, Highway 60 - F5 BOREHOLE TYPE Truck Mounted CME 45 - Hollow Stem Augers COMPILED BY SH

CLIENT AECOM DATE (Started) 18 September 2014 TIME 18 September 2014 (Completed) 6:00:00 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					
367.2	Ground Surface												
0.0	50 mm Asphalt 250 mm Crushed Gravel		1	SS	22								23 72 (5)
	FILL - sand some to with gravel trace to some silt trace clay mixed with rockfill		2	SS	38								
	brown		3	SS	25/0 mm								
	boulders encountered and cored over a length of 0.7 m at depths between 1.5 m and 2.3 m		4	SS	9								17 64 (19)
	(loose to dense)		5	SS	13								
363.5													
3.7	SAND some gravel some silt and clay		6	SS	36								
	(dense)		7	SS	39								15 69 (16)
361.6													
5.6	Auger Refual Start to rock coring		8	RC	Rec= 100% RQD= 35%								
	BEDROCK - grey gneiss		9	RC	Rec= 100% RQD= 59%								
	poor to fair quality												
358.5													
8.7	End of Sampling End of Borehole												

COMMENTS		+ 3, × 3 : Numbers on right refer to Sensitivity Numbers on left refer to values greater than 120 kPa		WATER LEVEL RECORDS	
Water level in borehole not stabilized.		○ 3% STRAIN AT FAILURE		Date (dd/mm/yy)/Time	Water Depth (m)
				1) 18/9/14 6:00:00 PM	2.31
				2)	-
				3)	-

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

MEL-GEO 14083 - BOREHOL LOGS - F5.GPJ MEL-GEO.GDT 27/3/15

METRIC**RECORD OF BOREHOLE NO. 3**

REFERENCE 14/07/14083 DATUM Geodetic LOCATION N 5023432 E 340897.4 - Franklin Twp., Station 13+027 ORIGINATED BY JL

PROJECT GWP 5333-11-00, Highway 60 - F5 BOREHOLE TYPE Track Mounted CME 45 - Hollow Stem Augers COMPILED BY SH

CLIENT AECOM DATE (Started) 25 August 2014 TIME (Completed) 1:00:00 PM CHECKED BY MAM

DATE (Completed) 25 August 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							
359.9	Ground Surface																		
0.0	Organic silty sand trace gravel 50 mm forest cover (very loose) dark brown		1	SS	2														
359.2	SAND gravelly with silt and clay grey (dense)		2	SS	39														
			3	SS	42														
357.8	Auger Refusal Start to rock coring BEDROCK - grey brown/pink gneiss good quality		4	RC	Rec=98% RQD=83%														
2.1			5	RC	Rec=95% RQD=89%														
354.7																			
5.2																			
COMMENTS								+ 3, x 3 : Numbers on right refer to Sensitivity Numbers on left refer to values greater than 120 kPa ○ 3% STRAIN AT FAILURE					WATER LEVEL RECORDS						
													<table border="1"> <thead> <tr> <th>Date (dd/mm/yy)/Time</th> <th>Water Depth (m)</th> <th>Cave In (m)</th> </tr> </thead> <tbody> <tr> <td>1) 25/8/14 1:00:00 PM</td> <td>0.33</td> <td>5.2</td> </tr> <tr> <td>2) 18/9/14 2:00:00 PM</td> <td>1.02</td> <td>-</td> </tr> <tr> <td>3)</td> <td>-</td> <td>-</td> </tr> </tbody> </table>				Date (dd/mm/yy)/Time	Water Depth (m)	Cave In (m)
Date (dd/mm/yy)/Time	Water Depth (m)	Cave In (m)																	
1) 25/8/14 1:00:00 PM	0.33	5.2																	
2) 18/9/14 2:00:00 PM	1.02	-																	
3)	-	-																	
The stratification lines represent approximate boundaries. The transition may be gradual.																			

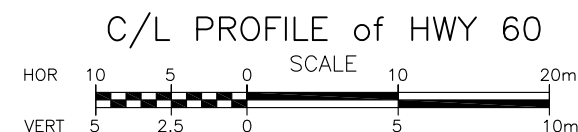
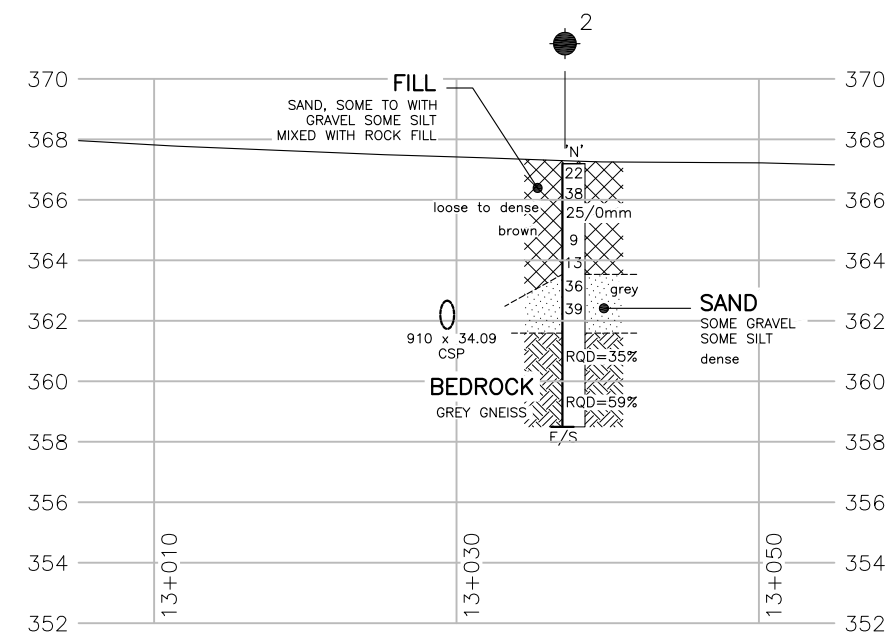
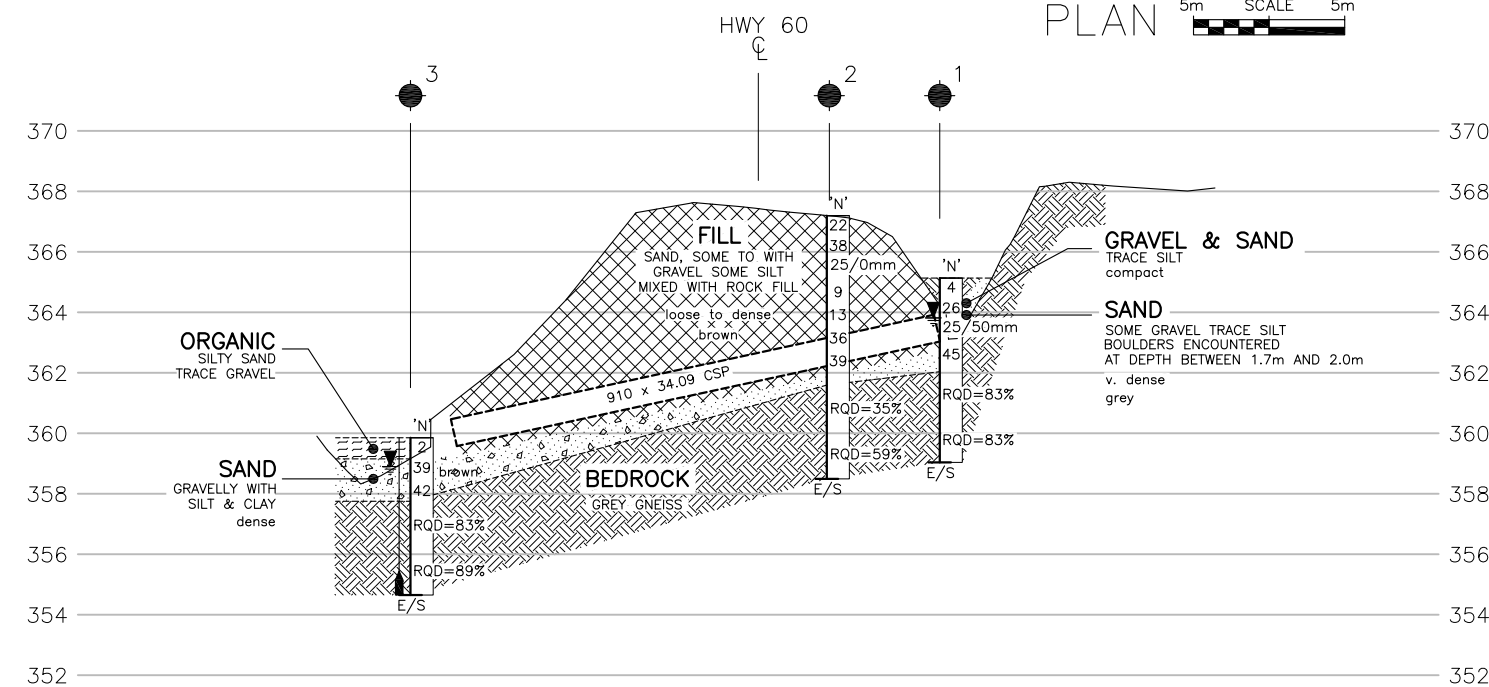
MEL-GEO 14083 - BOREHOL LOGS - F5.GPJ MEL-GEO.GDT 27/3/15

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

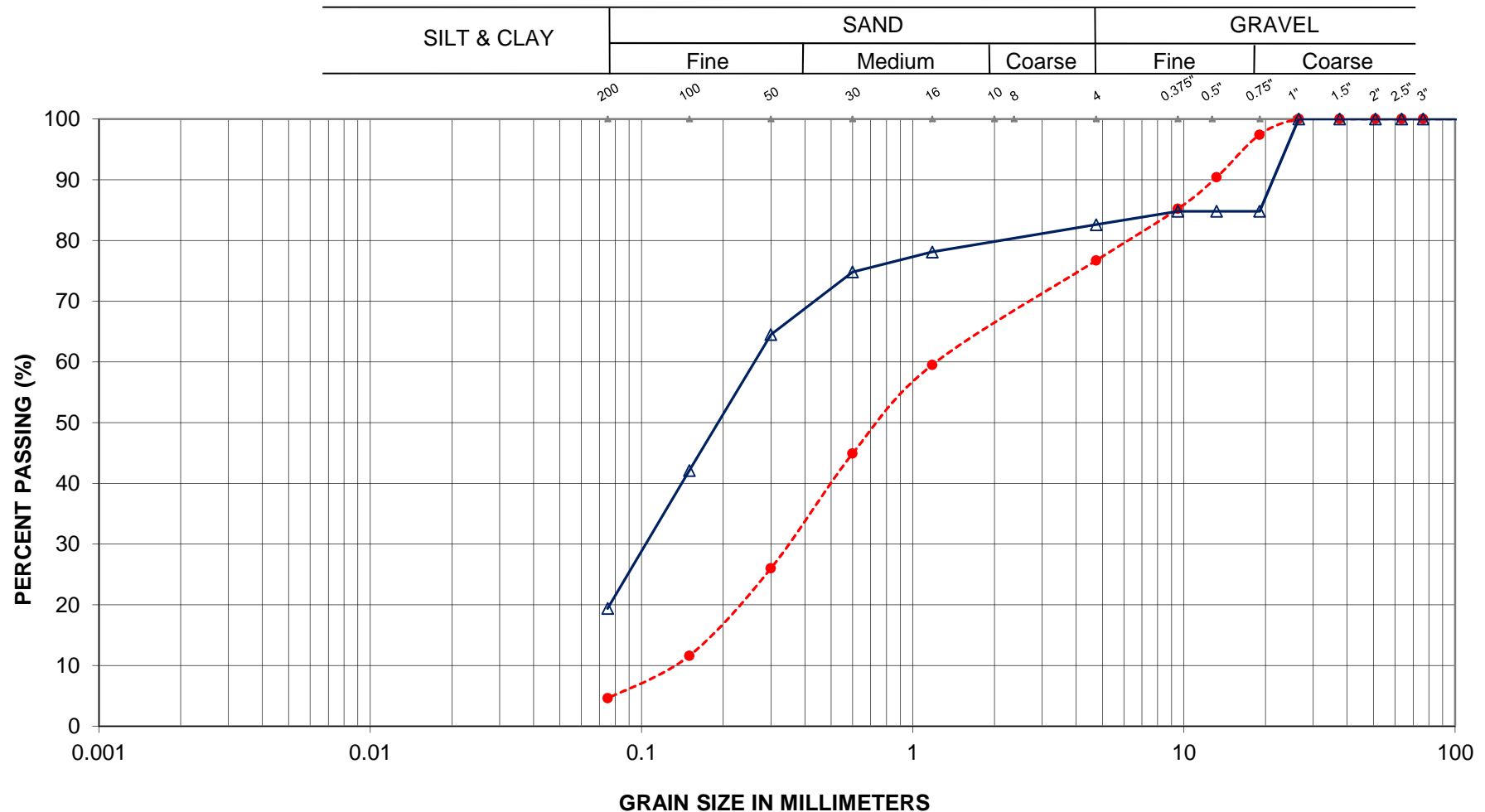
Appendix 3 Borehole Plan and Lab Data

Drawing No. 2: Borehole Location and Soil Strata
Figure Nos. L-1 to L-4: Grain Size Distribution Curves
Table No. L-5: Laboratory Test Summary Sheet



DRAWING NOT TO BE SCALED
50mm ON ORIGINAL DRAWING

REVISIONS	FEB/15	RG	DRAFT						
	MAR/15	RG	FINAL						
DESCRIPTION									
DESIGN			CHK			CODE	LOAD		DATE APR/15
DRAWN	RG	CHK	SH	SITE		STRUCT		SCHEME	DWG 2

GRAIN SIZE ANALYSIS

---◆--- BH No.: 2 Sa No.: 1 Depth: 0 - 0.61 m

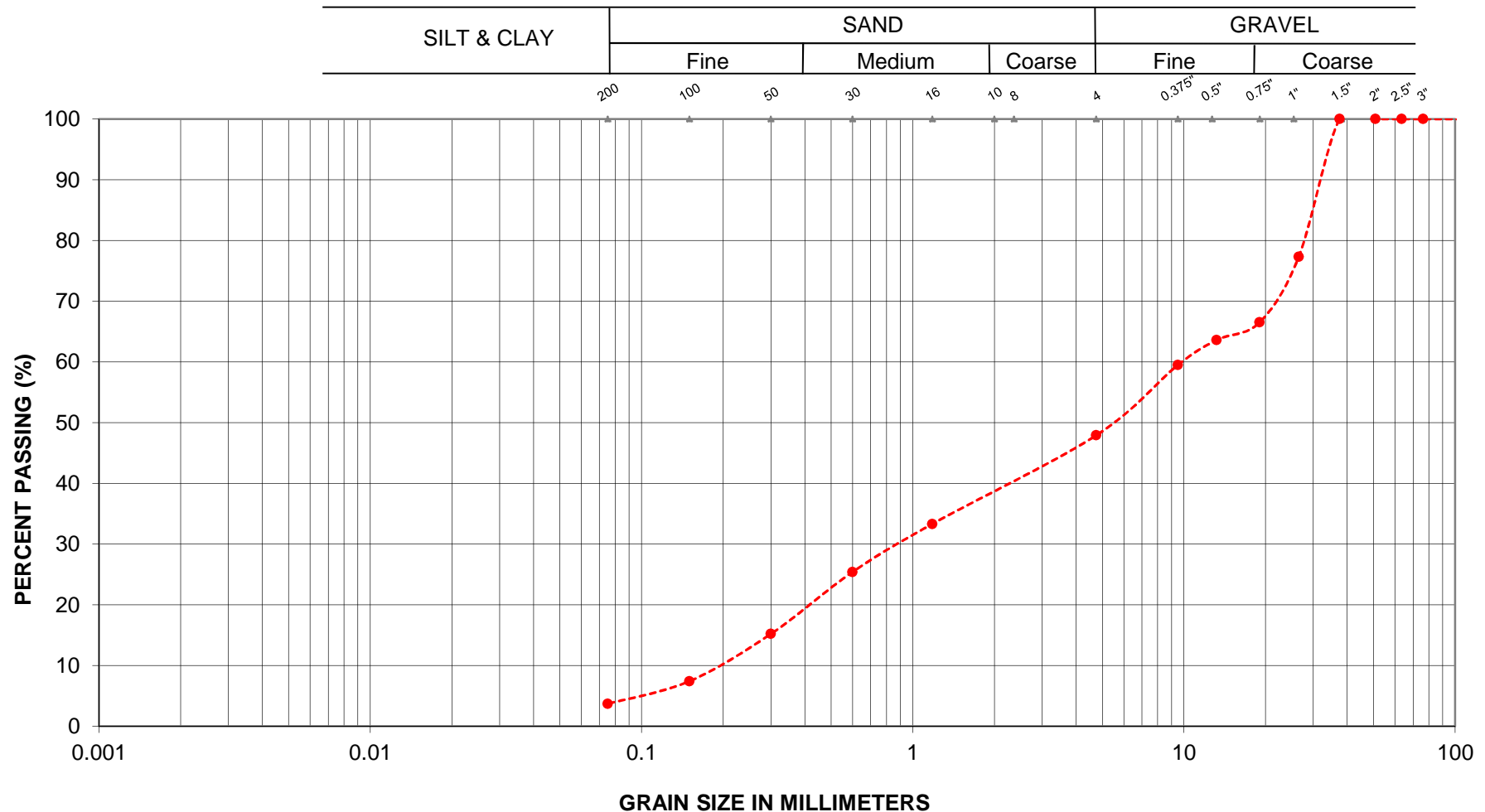
—▲— BH No.: 2 Sa No.: 4 Depth: 2.29 - 2.74 m

SAND FILL

LOCATION: Hwy 60 Sta. 13+029 Culvert
TWP Franklin, Ontario

LVM-Merlex, a Division EnGlobe Corp.

FIGURE L-1

GRAIN SIZE ANALYSIS

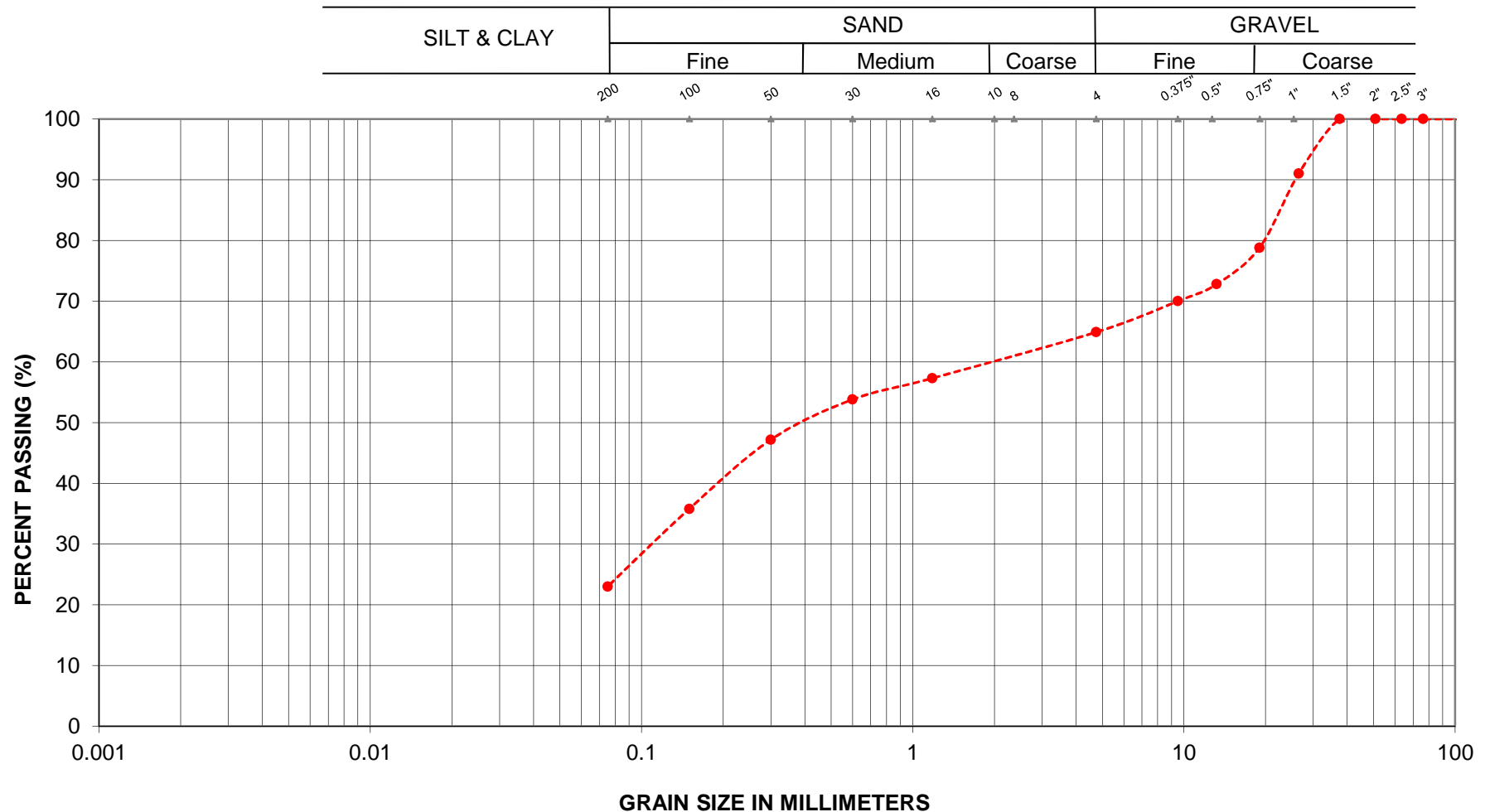
---●--- BH No.: 1 Sa No.: 2 Depth: 0.76 - 1.22 m

GRAVEL and SAND

LOCATION: Hwy 60 Sta. 13+029 Culvert
TWP Franklin, Ontario

LVM-Merlex, a Division EnGlobe Corp.

FIGURE L-2

GRAIN SIZE ANALYSIS

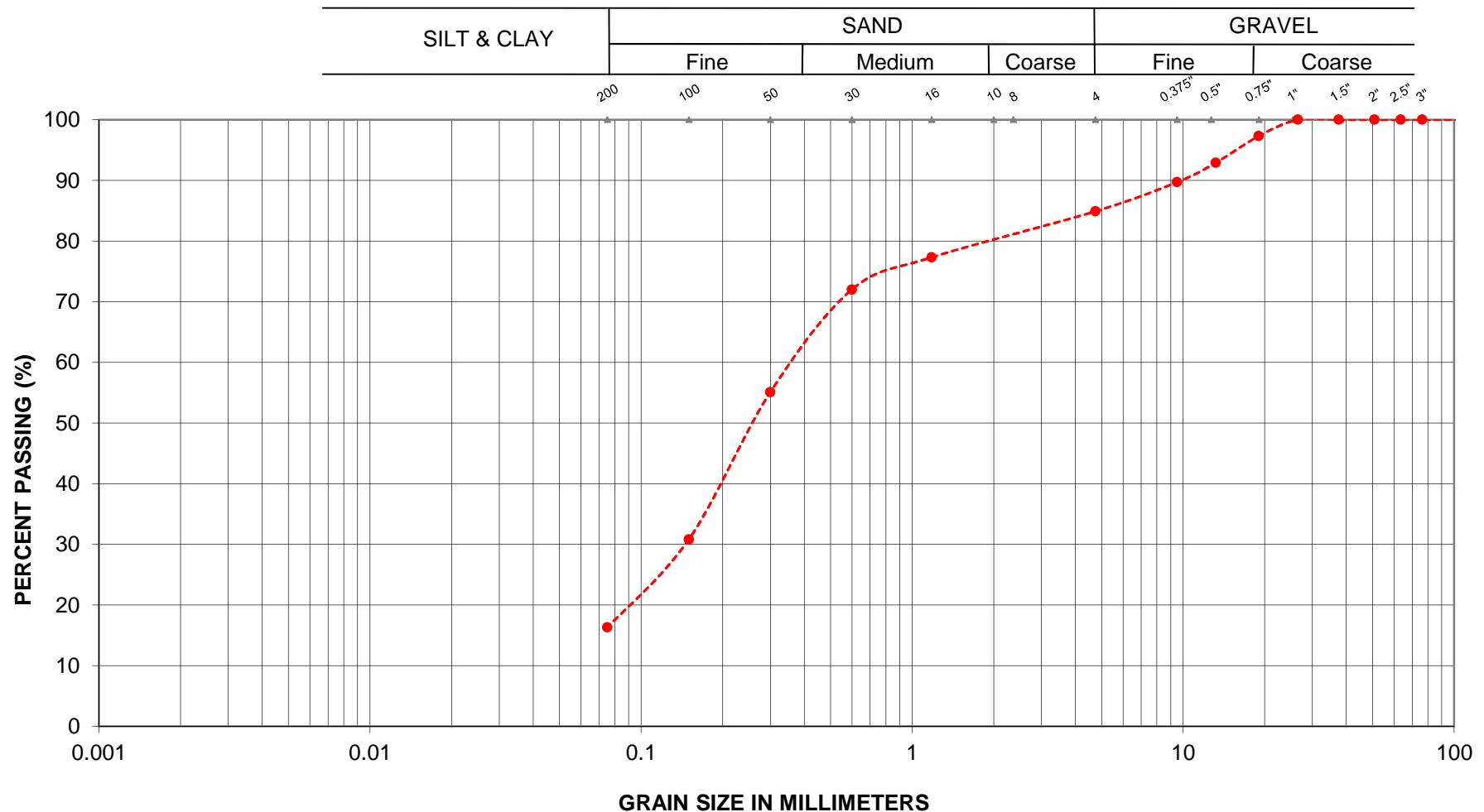
---●--- BH No.: 3 Sa No.: 3 Depth: 1.52 - 1.98 m

GRAVELLY SAND

LOCATION: Hwy 60 Sta. 13+029 Culvert
TWP Franklin, Ontario

LVM-Merlex, a Division EnGlobe Corp.

FIGURE L-3

GRAIN SIZE ANALYSIS

---●--- BH No.: 2 Sa No.: 7 Depth: 4.57 - 5.03 m

SAND

LOCATION: Hwy 60 Sta. 13+029 Culvert
TWP Franklin, Ontario

LVM-Merlex, a Division EnGlobe Corp.

FIGURE L-4

Laboratory Tests - Summary Sheet



Borehole No.	Sample No.	Depth	Grain Size Analysis				NMC	Atterberg Limits			SPT 'N'	USCS	Unit Weight (kN/m3)	Remarks
			Gravel Size (%)	Sand Size (%)	Silt Size (%)	Clay Size (%)		LL (%)	PL (%)	IP (%)				
1	1	0.3					15.2				4			
	2	1.0	52	44		4	11.1				26			
	3	1.8					19.0				25/51mm			
	4	2.5					11.2				45			
2	1	0.3					3.1				22			
	2	1.0	23	72		5	12.1				38			
	3	1.4									25/0 mm			No sample recovered
	4	2.5	17	64		19	15.0				9			
	5	3.3					25.4				13			
	6	4.0					25.8				36			
	7	4.8	15	69		16	9.5				39			
3	1	0.3					109.1				2			
	2	1.0					10.6				39			
	3	1.8	35	42		23	29.8				42			

Appendix 4 Photo Essay

Enclosure No. 5:

Photo Essay

Embankment at Culvert Location – Looking South

Photo: 1



Culvert Inlet – Looking West (Right Side)

Photo: 2



Project: Hwy 60 – Culvert 13+029

Photos Provided By: LVM

Date: August 2014

Culvert Outlet Location – Looking East (Left Side)

Photo: 3



Culvert Outlet – Looking West

Photo: 4



Project: Hwy 60 – Culvert 13+029

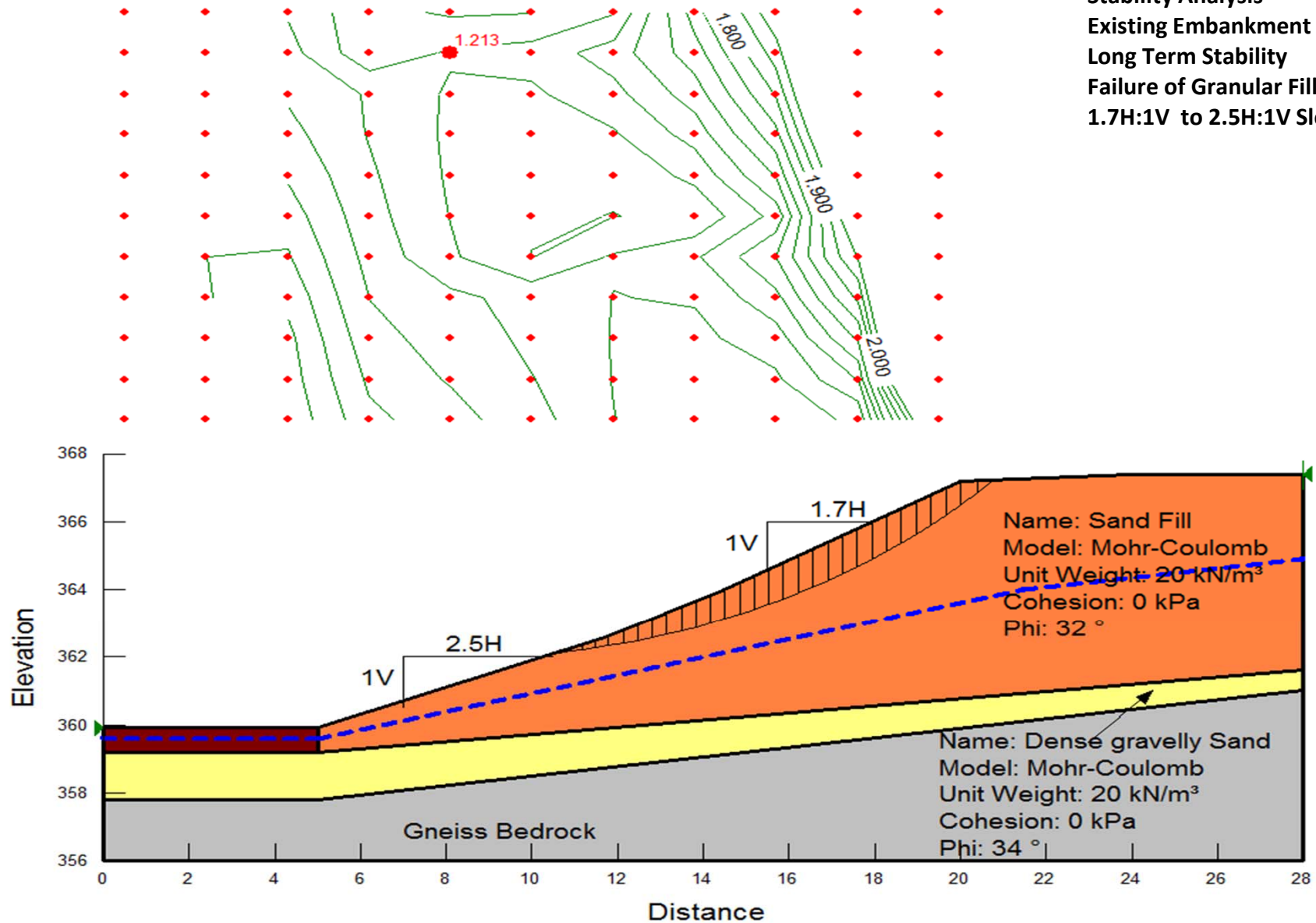
Photos Provided By: LVM

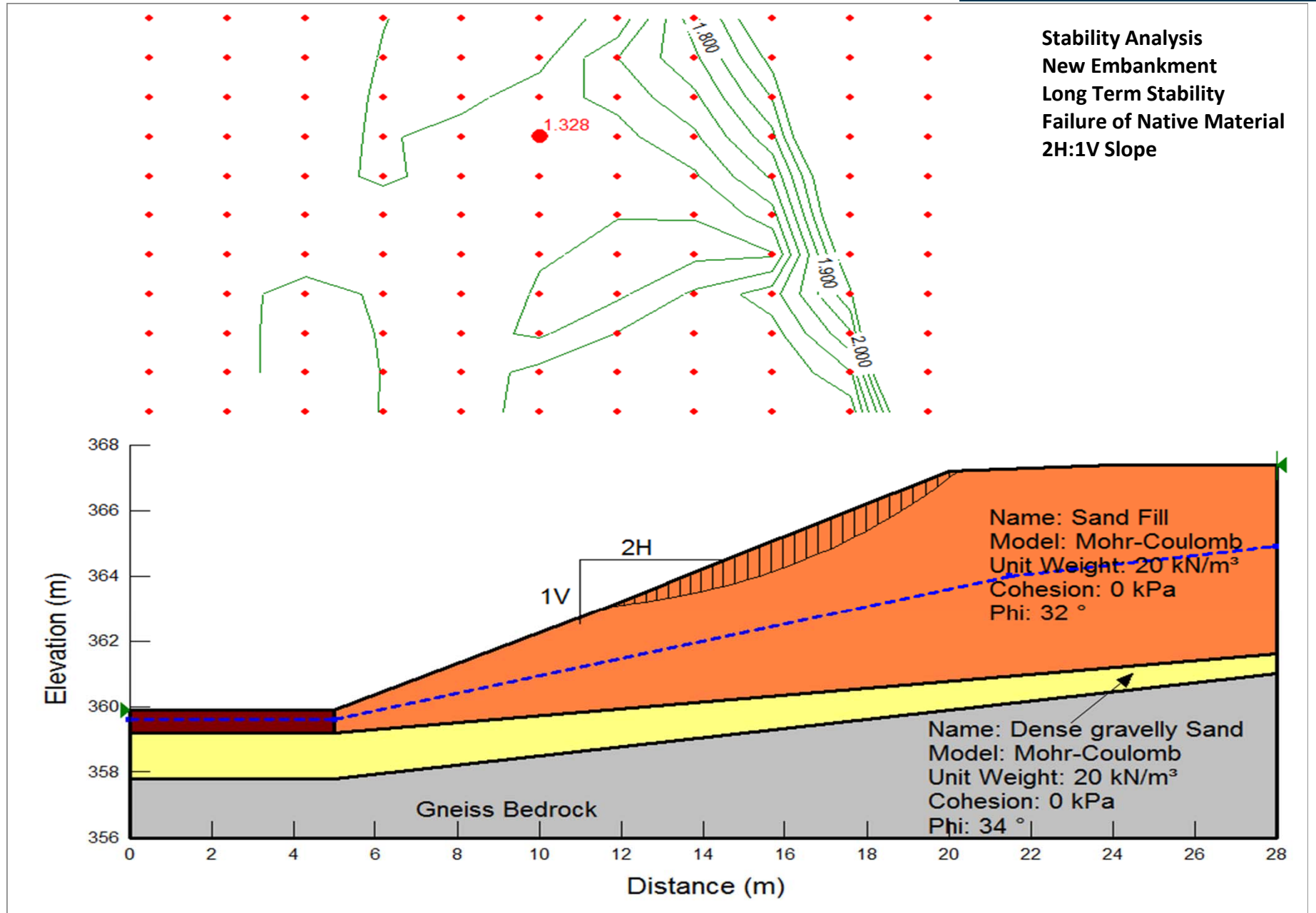
Date: August 2014

Appendix 5 Design Data

Figure Nos. S-1 and S-2:	Slope Stability Analyses
Table A:	Comparison of Shoring Alternatives
Figure No. SK-3:	Conceptual Staging Plan
Figure No. SK-4:	Conceptual Shoring Locations
Figure No. SK-5	Conceptual Shoring Sections

Stability Analysis
Existing Embankment
Long Term Stability
Failure of Granular Fill
1.7H:1V to 2.5H:1V Slopes

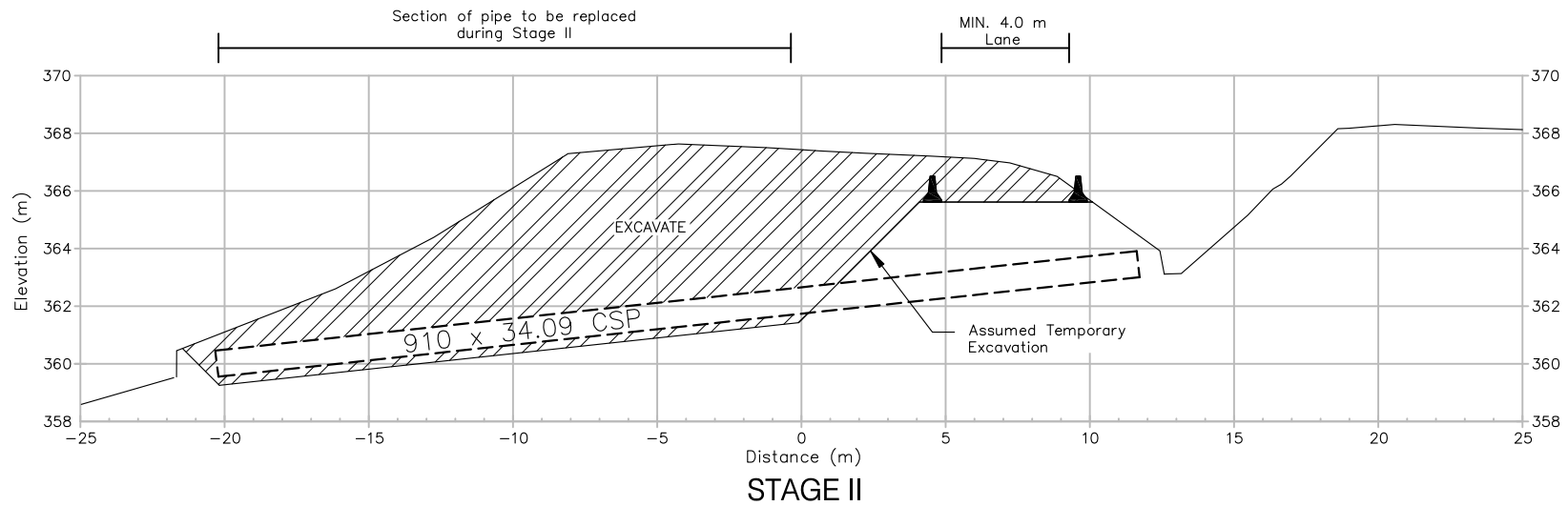
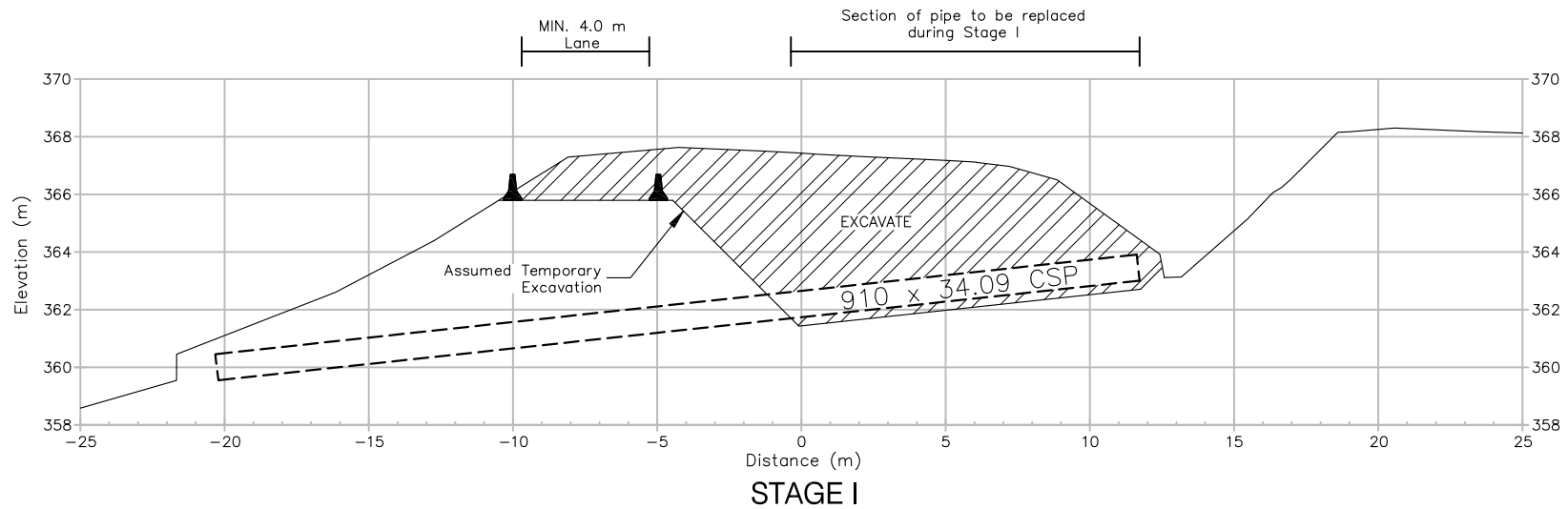




Date: March 2015

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 cobble/boulders present in fills and native soils.	\$ 650/m ²
Steel Sheet Piles	5 – 21	-High strength, continuous, -Readily available	-Limited by soil conditions (i.e. obstructions)	Not recommended due to required embedment in the bedrock	\$ 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	Feasible provided sufficiently socketed into the bedrock	
Tangent/ Secant/ Staggered Drilled Piles	10 – 18	-Readily available -Adaptable to various ground conditions	-Possible ground loss and/or seepage -Poor alignment tolerance	Feasible using special equipment drilled in the bedrock	
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	



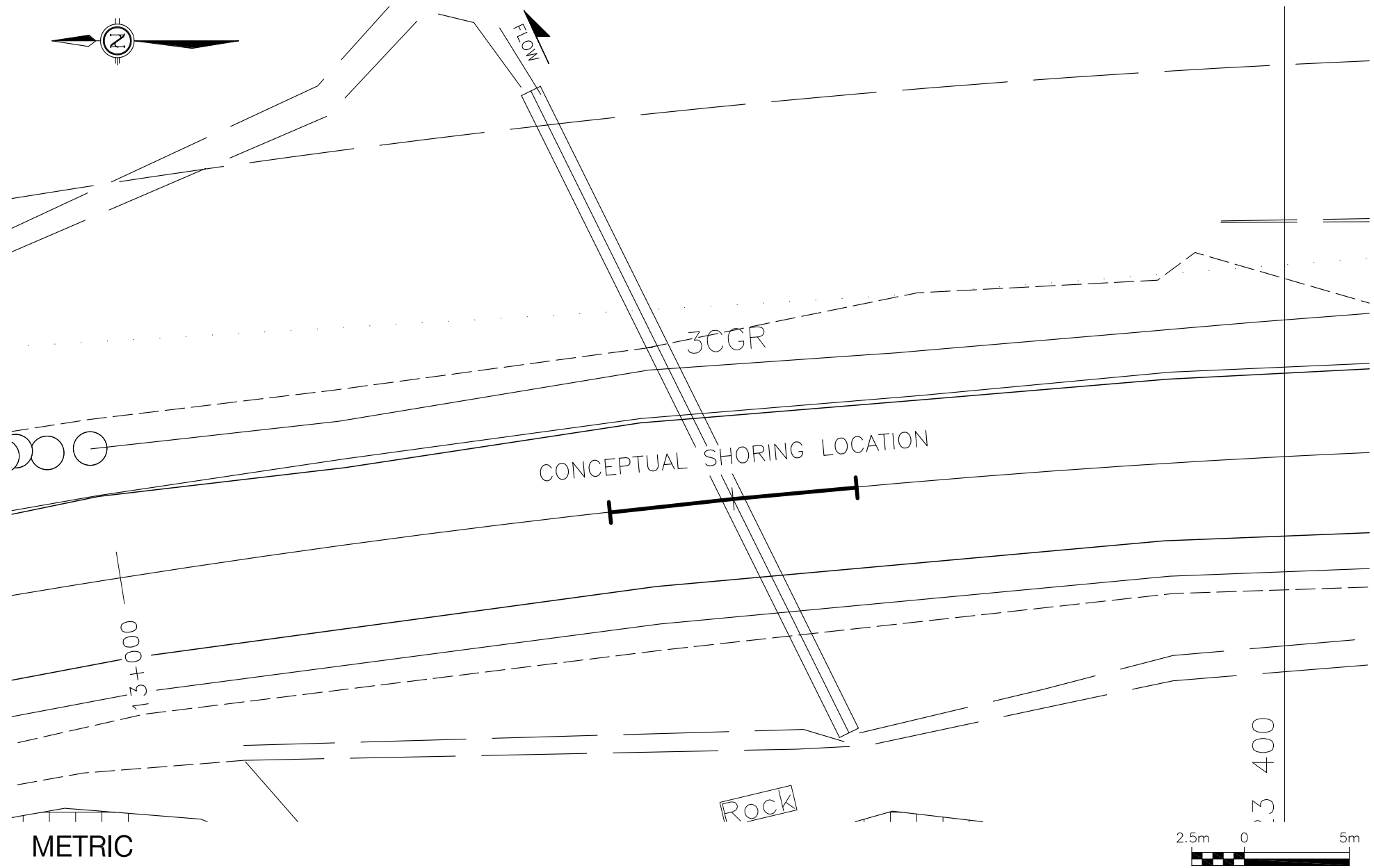
METRIC

Dimensions are in meters and/or millimeters unless otherwise shown. Stations are in kilometers + meters.



Highway 60, Township of Franklin - Culvert at Station 13+029
Conceptual Staging Cross-Sections

FIGURE SK-3

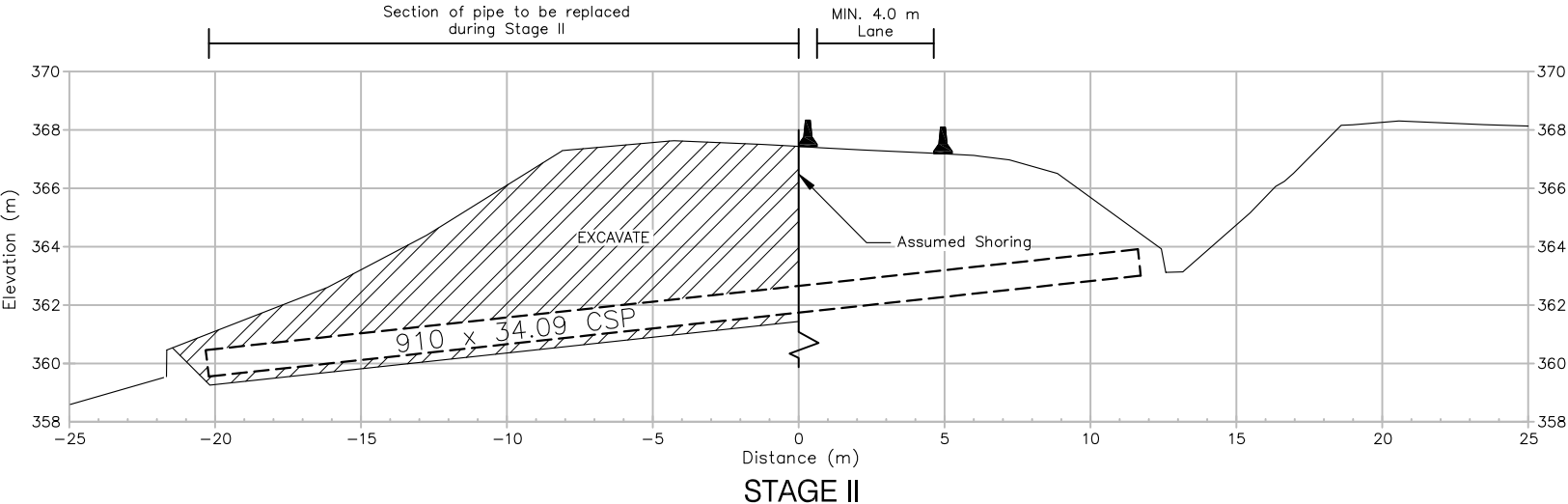
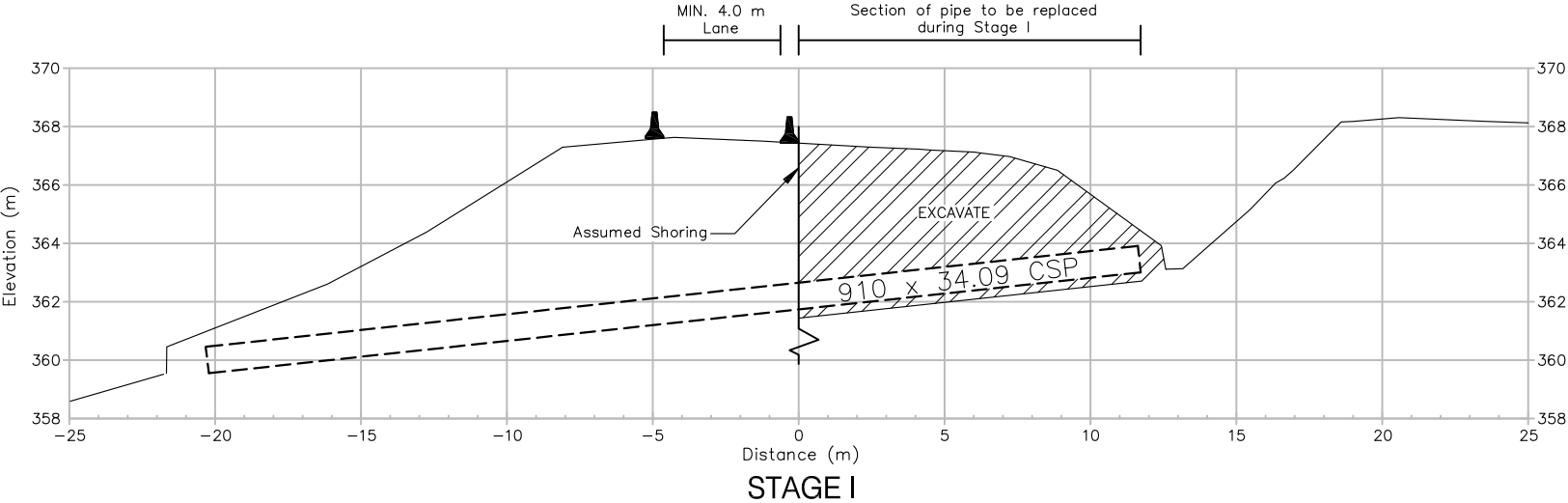


METRIC

Dimensions are in meters
and/or millimeters unless
otherwise shown. Stations are
in kilometers + meters.

Highway 60, Township of Franklin - Culvert at Station 13+029
Conceptual Shoring Location Plan

FIGURE SK-4



METRIC

Dimensions are in meters
and/or millimeters unless
otherwise shown. Stations are
in kilometers + meters.



Highway 60, Township of Franklin - Culvert at Station 13+029
Conceptual Shoring Cross-Section

FIGURE SK-5

