

**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 11+540 – Township of Sinclair
GWP 5333-11-00**

FINAL PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT

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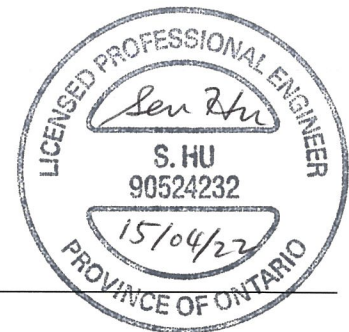
Final Preliminary Foundation Investigation and Design Report

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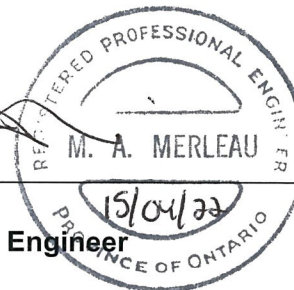


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1 INTRODUCTION

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 11+540 in the Township of Sinclair on Highway 60, some 1.3 km east of the intersection between Highway 60 and Harp Lake Road.

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 11+540 in the Township of Sinclair. The topography of this site is located in a valley area. The existing highway embankment currently supports two undivided lanes of highway, running in an east-west direction. The existing highway, at the culvert location, is constructed of a granular fill, mixed with rockfill, embankment some 8 m to 10.5 m in height, with centerline elevation of 298.8 m at the culvert location. The existing embankment slopes in the area of the culvert have been built between angles of approximately 1.85H:1V on the south side to 1.6H:1V to 3.6H:1V on the north side. The culvert at this location is a 1520 mm diameter Corrugated Steel Pipe (CSP) culvert, some 53 m in length. The flow through the culvert is from north to south (left to right).

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 18th and November 7th, 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 (east) and outlet (west) ends of the culverts.

The field investigation was carried out using a truck and bombardier mounted CME drilling rigs 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 boreholes prior to backfilling to allow for post borehole completion 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. L-4).

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 11+540, TWP OF SINCLAIR

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. 3 advanced through the embankment, and Borehole No. 2 advanced at the culvert outlet. At the time of the subsurface investigation, the ground surface elevations at Boreholes Nos. 1 to 3 inclusive were recorded at elevations 290.7 m, 289.1 m, and 298.8 m, respectively.

4.1.1 Pavement Structure

Borehole No. 3 was advanced through the embankment where a pavement structure consisting of 40 mm asphalt and 270 mm crushed gravel was penetrated. In addition, a separate 50 mm thick layer of asphalt was encountered at depth of 1.8 m below ground surface at the location of Borehole No.3.

4.1.2 Granular Fill

Underlying the pavement structure at Borehole No.3, a layer of fill consisting of brown sand trace to with gravel, trace to some silt, trace clay, and occasional cobble was penetrated. The natural moisture content measured on samples of this deposit was in the order of 3% to 17%. Gradation analyses were carried out on four (4) samples of this deposit, the results of which indicated 1% to 26% gravel size particles, 58% to 77% sand size particles, 12% to 26% silt and clay size particles (Figure No. L-1, Appendix 3). Based on SPT 'N' values of 11 to 35 blows per 30 mm penetration, the compactness of this deposit was described as compact to dense. This deposit was encountered to a depth of 10.2 m below grade at Borehole No. 3 (elevation 288.6 m).

4.1.3 Organic Soils

Below ground surface at Borehole No. 1, a layer of organic silty sand was penetrated. The natural moisture content measured on one (1) sample of this layer was some 49%. This organic soil layer was encountered to an approximate depth of 0.7 m below ground surface at Borehole No. 1 (elevation 290 m).

4.1.4 Sand

Underlying the granular fill at Borehole No.3 and the organic silty sand deposit at Borehole No. 1, a deposit of grey sand some to with gravel, some silt and clay, and occasional cobble was penetrated. The sand deposit was encountered at the ground surface at Borehole No. 2. The

natural moisture content measured on samples of this deposit was in the order of 8% to 32%. Gradation analysis was carried out on three (3) samples of this deposit, the results of which indicated 17% to 30% gravel size particles, 46% to 68% sand size particles, and 15% to 25% silt and clay size particles (Figure No. L-2, Appendix 3). Based on SPT 'N' values of 3 to 61 blows per 300 mm penetration, the compactness state of this deposit was described as very loose to very dense, generally compact. This deposit was encountered to depths of 2.0 m, 0.7 m and 13.3 m below grade at Borehole Nos. 1 to 3, respectively (elevations 288.7 m, 288.4 m and 285.6 m, respectively).

4.1.5 Sand and Gravel

Underlying the sand at Borehole No. 2, a deposit of brown sand and gravel trace silt was penetrated. The natural moisture content measured on samples of this deposit was in the order of 17% to 18%. The gradation analysis was carried out on one (1) sample of this deposit, the results of which indicated 40% gravel size particles, 51% sand size particles, and 9% silt and clay size particles (Figure No. L-3, Appendix 3). Based on SPT 'N' values of 48 and 47 blows per 300 mm penetration, the compactness state of this deposit was described as dense state. This deposit was encountered to a depth of 2 m below grade at Borehole No. 1 (elevation 288.7 m).

4.1.6 Bedrock

Underlying the above described sand and gravel deposit at Borehole No. 2 and sands at Borehole Nos. 1 and 3, bedrock was proven by diamond core drilling. The bedrock was described as greyish gneiss. Based on values of Rock Quality Designation (RQD) ranging from 53% to 96%, the bedrock was described as fair to excellent quality. Sampling in the bedrock was terminated at depths of 5 m, 5.5 m, and 16.3 m below grade at Borehole Nos. 1 to 3, respectively (elevations 285.7 m, 283.6 m, and 282.5 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 the investigation period (November 5 to November 7th, 2014), the creek water levels were measured at elevations some 290.1 m at the inlet area and 288.4 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. Standpipes were respectively installed in Borehole Nos. 1 and 3 to obtain post borehole completion water levels. These levels are recorded on the individual Record of Borehole Log Sheets (Appendix B).

The groundwater levels were measured at elevations 290.7 m and 290.2 m at Borehole Nos. 1 and 3, respectively.

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

5 DISCUSSION AND RECOMMENDATIONS

5.1 GENERAL

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

The existing culvert, located at Station 11+540, in the Township of Sinclair, is a 1520 mm diameter CSP culvert some 53 m long. The invert depths of existing culvert range from some 1.6 m (elevation 290.8 m) at the inlet (left side) to some 1.5 m below ground surface (elevation 288.4 m) at the outlet (right side). The invert of the existing culvert at centerline is at a depth of some 9.8 m (elevation 289.2 m). The existing highway embankment currently supports two undivided lanes of highway, running in a east-west direction. The flow through the existing culvert is from left to right (east to west). 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 compact to dense sands to sands and gravels.

The type of culvert (concrete, CSP, or High Density Polyethylene (HDPE)) to replace the existing culverts is currently unknown. It is assumed that the new culvert will be constructed along a similar skew and vertical alignment as the existing condition.

5.2 FOUNDATION CONSIDERATIONS

The founding native dense sands overlying the bedrock present below the existing embankment are considered adequate for support of a culvert and for a conventional highway embankment of this height. Bearing resistance should not be a major issue provided the natural bearing surface is not unduly disturbed during construction and groundwater is controlled throughout construction, as discussed in Section 5.5. Adequate dewatering is required to avoid potential boiling and disturbance of subgrade at the founding level.

Based on the characteristics of the native sand to sand and gravel subgrades present below the culvert, the response of the existing embankment, and a founding elevation similar to that of the existing culverts, a factored bearing resistance at ULS of 450 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 350 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 8 m on the northern side slope and 10.5 m on the south side slope. The angles of existing slopes are some 3.6H:1V (above approximate elevation 296 m) to 1.6H:1V (below approximate elevation 296 m) on the north slope and 1.85H:1V on the south 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 north and the south slopes with existing inclinations in the granular fill, respectively. For the purposes of these analyses, the materials were modeled using the following parameters;

PARAMETER	MATERIAL		
	EMBANKMENT GRANULAR FILL	SAND	SAND AND GRAVEL
Unit Weight (kN/m ³)	19	19	20
Effective Friction Angle (degrees)	32	32	33

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 1.0 on the existing north and 1.2 on south slopes of embankment except lower values against the minor surficial slippage on the 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.

A 50 mm thick old asphalt was encountered at a depth of 1.8 m below ground surface (i.e. elevation 297 m) at location of Borehole No. 3. Based on Drawing 2 in Appendix 3, it is likely that the previous embankment was raised above some Elevation 297 m in the past to an Elevation 298.8 m at the centerline of the highway at the existing culvert area. Vegetation and rock fragments likely rockfilled material were observed on the existing slopes as shown on Enclosure No.5; therefore the surficial slippage is not likely an issue for the long term stability. It is recommended that the lower northern slope of embankment be cut to not steeper than 2H:1V. Based on OPSD 202.010, a bench of 2 m minimum width will be required for the 2H:1V slope of the earth embankment greater than 8 m in height. The long term stability of the new embankment slopes 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 compact to dense sands to sand and gravel. 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 long term 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 material of bedding and cover shall be placed in uniform layers not exceeding uncompacted thickness of 200 mm. The elevation difference of backfilling on either side of the rigid pipe shall be limited to a maximum 200 mm per OPSS 401. 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. 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. 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 MTO-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 only 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 not be used at this culvert location.

5.3.2 Flexible Culvert

Flexible culverts (i.e. CSP/SPCSP/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 embedment material shall be placed in uniform layers not exceeding uncompacted thickness of 200 mm. The elevation difference of the embedment fill on either side of the flexible pipe must be limited to a maximum 200 mm per OPSS 401. The backfill should be placed to a minimum depth of 900 mm above the crown of the pipe before power tractors or rolling equipment can be used for compacting per OPSS 401.

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 8 m below the edge of the northern embankment at the inlet (elevation 289.8 m) to some 10.5 m below the edge of the southern embankment at the outlet (elevation 288.4 m). The invert of the existing culvert at centerline is at a depth of some 9.8 m (elevation 289 m). Accordingly a minimum 8.3 m deep excavation (i.e. to elevation 289.5 m) at the northern slope and 10.8 m deep excavation (i.e. to elevation 288.1 m) at the southern 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 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.

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 a 24/7 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 296.8 m.
- Limit traffic to a single lane on the left (north), with a minimum platform width of 4 m, under 24/7 traffic control.
- Open cut excavate, to the right (south), and install approximately 28 m in length of new culvert.
- Reconstruct the embankment on the right (south), with a minimum platform width of 4 m for traffic.
- Divert the single lane of traffic to the right and continue open excavation to install the remainder of the culvert on the left (north).
- As the width of the platform increases on the left, 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 10 m of granular fills. The embankment fill is generally underlain by some 1.5 m to 3 m thick compact to dense sands to very dense sand and gravel deposits underlain by the bedrock. As noted rock fill mixed granular fill was encountered at depths from some 4.6 m to 5.2 m below ground surface in the embankment. As such, a temporary vertical wall for a protection system can likely consist of concrete caissons or micropiles with at least 1 m embedded depth in the bedrock. Alternatively a sufficiently robust steel sheet pile wall may be considered if additional investigation warrants. Conceptual shoring locations and sections 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 5.6,

γ = unit weight, as described in Section 5.6, 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. A conceptual shoring location is illustrated on Figure No. SK-4 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.

5.4.3 Trenchless/Tunnelling Techniques

The embankment at this location is some 8 m to 10.5 m in heights above the culvert invert. As such, culvert replacement using traditional open cut excavation with staged construction and/or temporary protection shoring wall may not be considered as economical methods at this location. Additional alternative methods using trenchless/tunnelling techniques for the culvert replacement are discussed in the following.

Borehole No. 3 through the embankment indicates that random rock fill, containing cobble sized rock fragments, has been used to construct the existing embankment at Station 11+540. The embankment is up to 10.5 m in height on the southern side slope. As such a trenchless approach for culvert replacement would eliminate the need for a detour and associated traffic delays. Several trenchless technologies are available, as outlined in the following Table B in Appendix 5; however the potential mixed rock fill composition of the embankment at the culvert location will limit the type of trenchless method that can be used at this site.

As noted above, the existing CSP culvert is 1520 mm in diameter. The culvert replacement may be considered to consist of lining the existing culvert and advancing a secondary culvert, some 1 m or less diameter, parallel to the existing, to accommodate the reduced capacity associated with lining of the existing culvert.

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 sizes and were in a generally compact 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 levels in the creek were recorded at elevations some 290.1 m at the inlet area and 288 m at the outlet area during the period of this investigation and the groundwater level in Borehole No. 3 (at the southern area of the embankment) had stabilized at elevation 290.2 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 would 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, and would be complicated due to the construction of the detour. 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 shallow depths ranging from 1.4 m to 3.5 m below the existing culvert invert. Depending on the invert and type of new culvert, the bedrock may not be within the anticipated depth of excavation; therefore bedrock excavation and/or blasting operations may not be required.

If blasting is required reference shall be made to OPSS 120. A blast design would be provided by the blasting contractor if the blasting operations are required.

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	SAND AND GRAVEL
Unit Weight (kN/m ³)	22	21	19	19	20
Angle of Internal Friction	34°	33°	32°	32°	33°
Coefficient of Active Earth Pressure (K_a)	0.28	0.29	0.31	0.31	0.29
Coefficient of Passive Earth Pressure (K_p)	3.54	3.39	3.23	3.23	3.39
Coefficient of Earth Pressure at Rest (K_o)	0.44	0.46	0.47	0.47	0.46

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. As noted in Section 5.5 the culvert subgrade must be adequately dewatered to maintain the bearing resistance of the foundation subgrade.

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 5024782.9 E 335957.9 - Sinclair Twp., Station 11+538 ORIGINATED BY JL
 PROJECT GWP 5333-11-00, Highway 60 - F4 BOREHOLE TYPE Track Mounted CME 45 - Hollow Stem Augers COMPILED BY SH
 CLIENT AECOM DATE (Started) 2014 November 07 TIME 2014 November 07
 DATE (Completed) 2014 November 07 (Completed) 12:10:00 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																				
290.7	Ground Surface																								
0.0	Organic silty sand trace gravel occasional grasslets dark brown (very loose)		1	SS	1																				
290.0																									
0.7	SAND some gravel some silt and clay occasional cobble (compact/dense) grey		2	SS	11																				
288.7			3	SS	38																				
2.0	Auger Refusal Start to rock coring																								
	BEDROCK - grey gneiss fair quality		4	RC	Rec= 100% RQD= 53%																				
			5	RC	Rec= 100% RQD= 73%																				
285.7																									
5.0	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 ○ 3% STRAIN AT FAILURE																	
								WATER LEVEL RECORDS <table border="1"> <thead> <tr> <th>Date (yy/mm/dd)/Time</th> <th>Water Depth (m)</th> <th>Cave In (m)</th> </tr> </thead> <tbody> <tr> <td>1) 14/11/07 12:00:00 PM</td> <td>0</td> <td>3.35</td> </tr> <tr> <td>2) 14/11/07 12:10:00 PM</td> <td>0</td> <td>-</td> </tr> <tr> <td>3)</td> <td>-</td> <td>-</td> </tr> </tbody> </table>						Date (yy/mm/dd)/Time	Water Depth (m)	Cave In (m)	1) 14/11/07 12:00:00 PM	0	3.35	2) 14/11/07 12:10:00 PM	0	-	3)	-	-
Date (yy/mm/dd)/Time	Water Depth (m)	Cave In (m)																							
1) 14/11/07 12:00:00 PM	0	3.35																							
2) 14/11/07 12:10:00 PM	0	-																							
3)	-	-																							

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

LVM-Merlex, a Division of EnGlobe Corp.

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

MEL-GEO 14083 - BOREHOLE LOGS - F4.GPJ MEL-GEO.GDT 15/01/19

METRIC**RECORD OF BOREHOLE NO. 2**

REFERENCE 14/07/14083 DATUM Geodetic LOCATION N 5024755.5 E 335911.0 - Sinclair Twp., Station 11+532 ORIGINATED BY JL
 PROJECT GWP 5333-11-00, Highway 60 - F4 BOREHOLE TYPE Track Mounted CME 45 - Hollow Stem Augers COMPILED BY SH
 CLIENT AECOM DATE (Started) November 6, 2014 TIME (Completed) 12:00:00 PM CHECKED BY MAM
 DATE (Completed) November 6, 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			SHEAR STRENGTH kPa									
							20	40	60	80	100						
289.1	Ground Surface		1	SS	3												
0.0	SAND with gravel some silt and clay occasional grass rootlets dark brown (very loose)															30 46 (24)	
288.4			2	SS	48												
0.7	SAND and GRAVEL trace silt and clay rock fragments of cobble size encountered (dense) brown															40 51 (9)	
			3	SS	47												
286.9																	
2.2	Auger Refusal Start to rock coring																
	BEDROCK - grey gneiss good quality		4	RC	Rec=100% RQD=82%												
			5	RC	Rec=100% RQD=78%												
			6	RC	Rec=100% RQD=75%												
283.6																	
5.5	End of Sampling End of Borehole																

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

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

MEL-GEO 14083 - BOREHOLE LOGS - F4.GPJ MEL-GEO.GDT 4/21/15

METRIC

RECORD OF BOREHOLE NO. 3



REFERENCE 14/07/14083 DATUM Geodetic LOCATION N 5024767.4 E 335927.7 - Sinclair Twp., Station 11+532.4 ORIGINATED BY TB
 PROJECT GWP 5333-11-00, Highway 60 - F4 BOREHOLE TYPE Truck Mounted CME 75 - Hollow Stem Augers COMPILED BY SH
 CLIENT AECOM DATE (Started) 2014 August 18 TIME 2014 August 18 (Completed) 9:30:00 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								
298.8	Ground Surface												
0.0	40 mm Asphalt 270 mm Crushed Gravel FILL - sand trace to with gravel trace to some silt trace clay		1	SS	20								
			2	SS	21								26 62 (12)
	50 mm Asphalt encountered at depth of 1.8 m dark greyish trace organic encountered at depth of 3 m brown to grey		3	SS	24								
			4	SS	35								
			5A	SS	35								9 73 (17)
			5B	SS									
	cobble / rockfill encountered at depths from 4.6 m to 5.2 m		6	SS	20								
			7	SS	18								16 58 19 7
			8	SS	12								
			9	SS	11								
			10A	SS	16								1 77 (22)
			10B	SS									
	seam of silt encountered at depth of 9.1 m		11	SS	5								
	gravel and sand encountered at depths from 9.9 m to 10.2 m (compact/very dense)		12	SS	5								
288.6			13A	SS	66								
10.2	SAND with gravel some silt and clay		13B	SS									
	grey (dense/very dense)		14	SS	31								25 50 (25)
			15	SS	61								
	Continued Next Page												

COMMENTS		WATER LEVEL RECORDS		
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		Date (yy/mm/dd)/Time	Water Depth (m)	Cave In (m)
		1) 14/08/26 10:30:00 AM	8.56	▽
		2)	-	▽
		3)	-	▽

MEL-GEO 14083 - BOREHOLE LOGS - F4.GPJ MEL-GEO GDT 15/01/19

METRIC**RECORD OF BOREHOLE NO. 3**

REFERENCE 14/07/14083 DATUM Geodetic LOCATION N 5024767.4 E 335927.7 - Sinclair Twp., Station 11+532.4 ORIGINATED BY TB
 PROJECT GWP 5333-11-00, Highway 60 - F4 BOREHOLE TYPE Truck Mounted CME 75 - Hollow Stem Augers COMPILED BY SH
 CLIENT AECOM DATE (Started) 2014 August 18 TIME
 DATE (Completed) 2014 August 18 (Completed) 9:30:00 PM CHECKED BY MAM

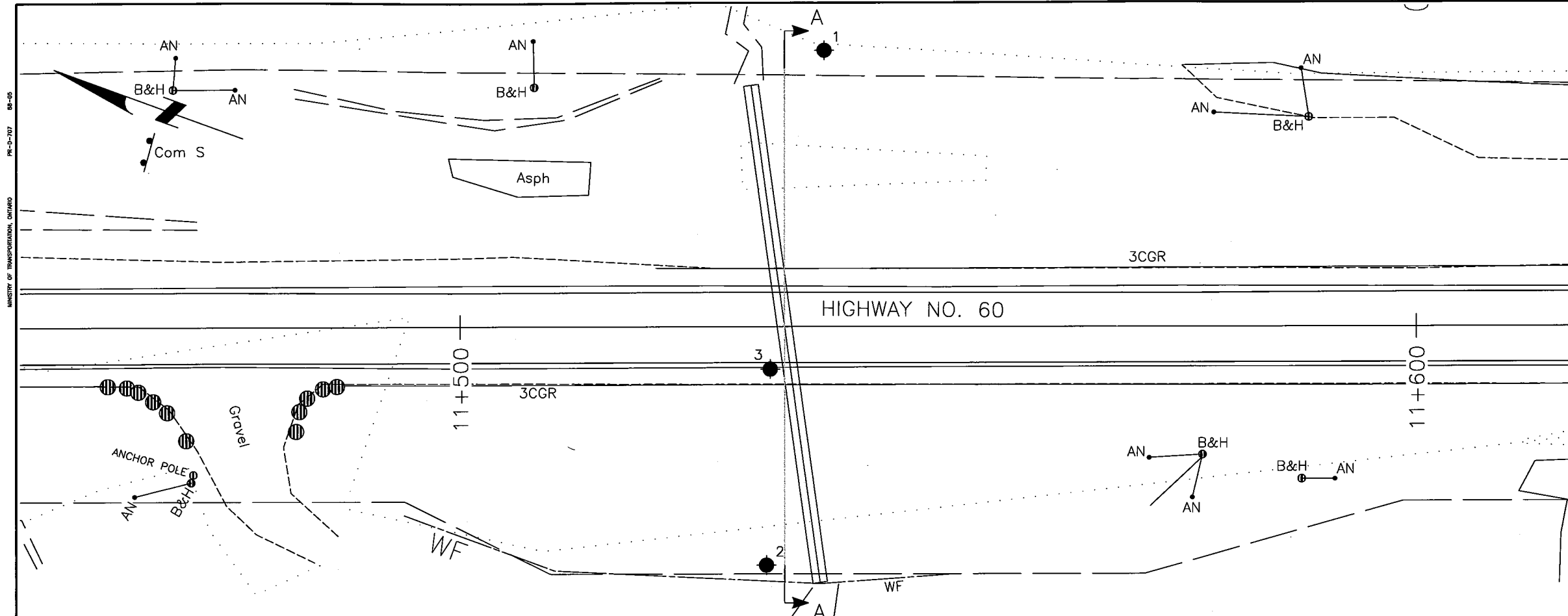
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100	W _p	W	W _L		
285.6	Continued from Previous Page																
13.3	Auger Refual Start to rock coring BEDROCK - grey gneiss excellent quality		16	RC	Rec=100% RQD=95%												
			17	RC	Rec=96% RQD=96%												
282.5	End of Sampling End of Borehole																
16.3																	

MEL-GEO 14083 - BOREHOL LOGS - F4.GPJ MEL-GEO.GDT 15/01/19

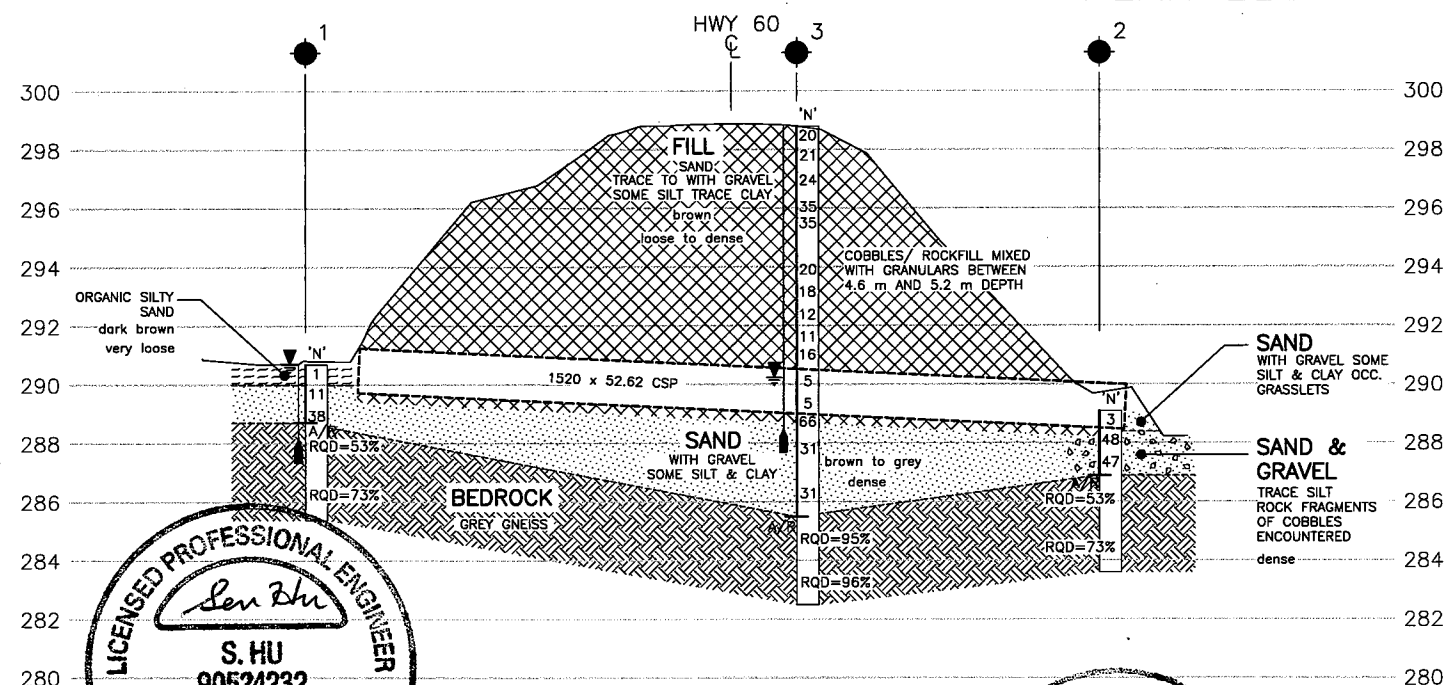
Appendix 3 Borehole Plan and Laboratory Data

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

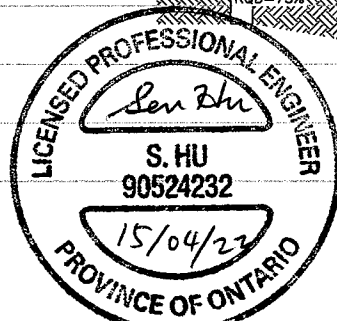
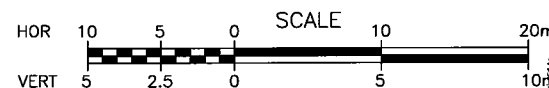
CAD FILE LOCATION AND NAME: \\B01\14083 - PAV & FDN, Hwy 60, Huntsville & Hwy 117, Cobalt\AECOM\FOUNDATIONS\Drawings\F4\Working - Do Not Delete Any Files\14083-F4 - Borehole Location Plan, 11+540 Sinclair Township
MODIFIED: 26/02/2015 11:29:22 AM BY: GRASRY
DATE PLOTTED: 26/02/2015 11:31:45 AM BY: RYAN GRASSER



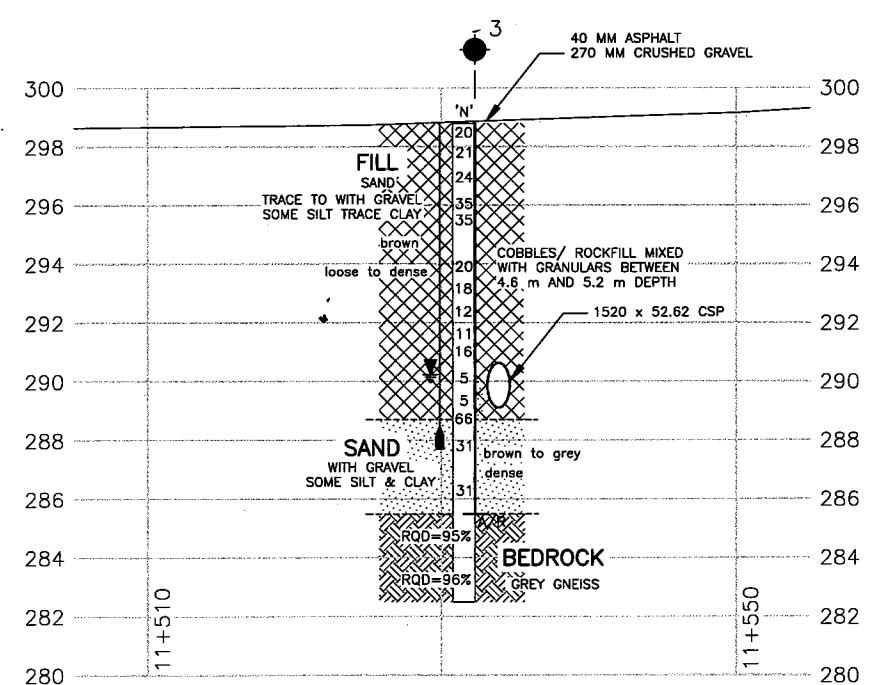
PLAN 5m SCALE 5m



SECTION A-A



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

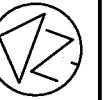


C/L PROFILE of HWY 60



DRAWING NOT TO BE SCALED
50mm ON ORIGINAL DRAWING

DISTRICT
CONT. No.
GWP No. 5333-11-00



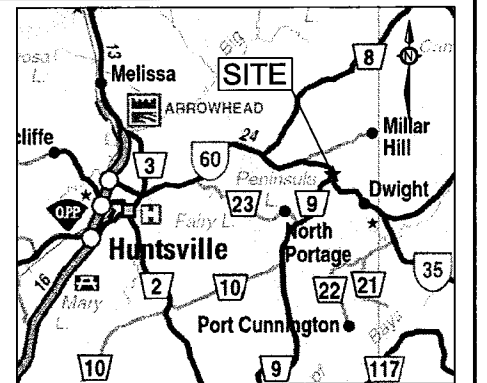
HWY 60
CULVERT AT STATION 11+534
SINCLAIR TOWNSHIP
BOREHOLE LOCATIONS
AND SOIL STRATA

DRAWING

2

LVM Merlex

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	290.7	29.0m Lt	5024782.9	345957.9
2	289.1	25.0m Rt	5024755.5	345911.0
3	298.8	4.5m Rt	5024767.4	345927.7

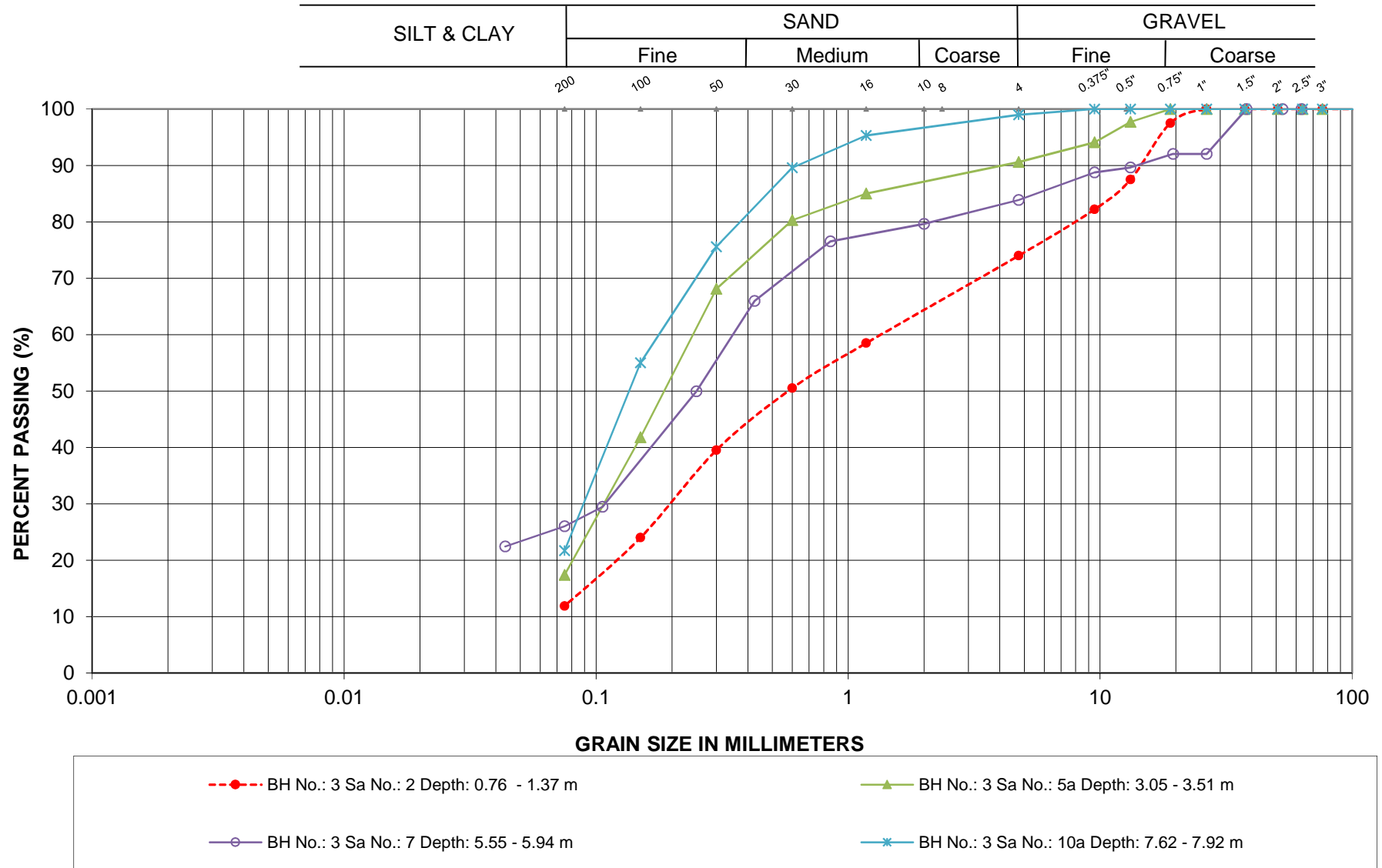
NOTES:

- The boundaries between soil strata have been established at the borehole locations only. The boundaries illustrated and stratigraphy between boreholes on this drawing are assumed based on borehole data and may vary. They are intended for design only.
- Base plan and alignment provided in digital format by exp. on October 22, 2014.

GEOCRES No. 31E-350

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

GRAIN SIZE ANALYSIS

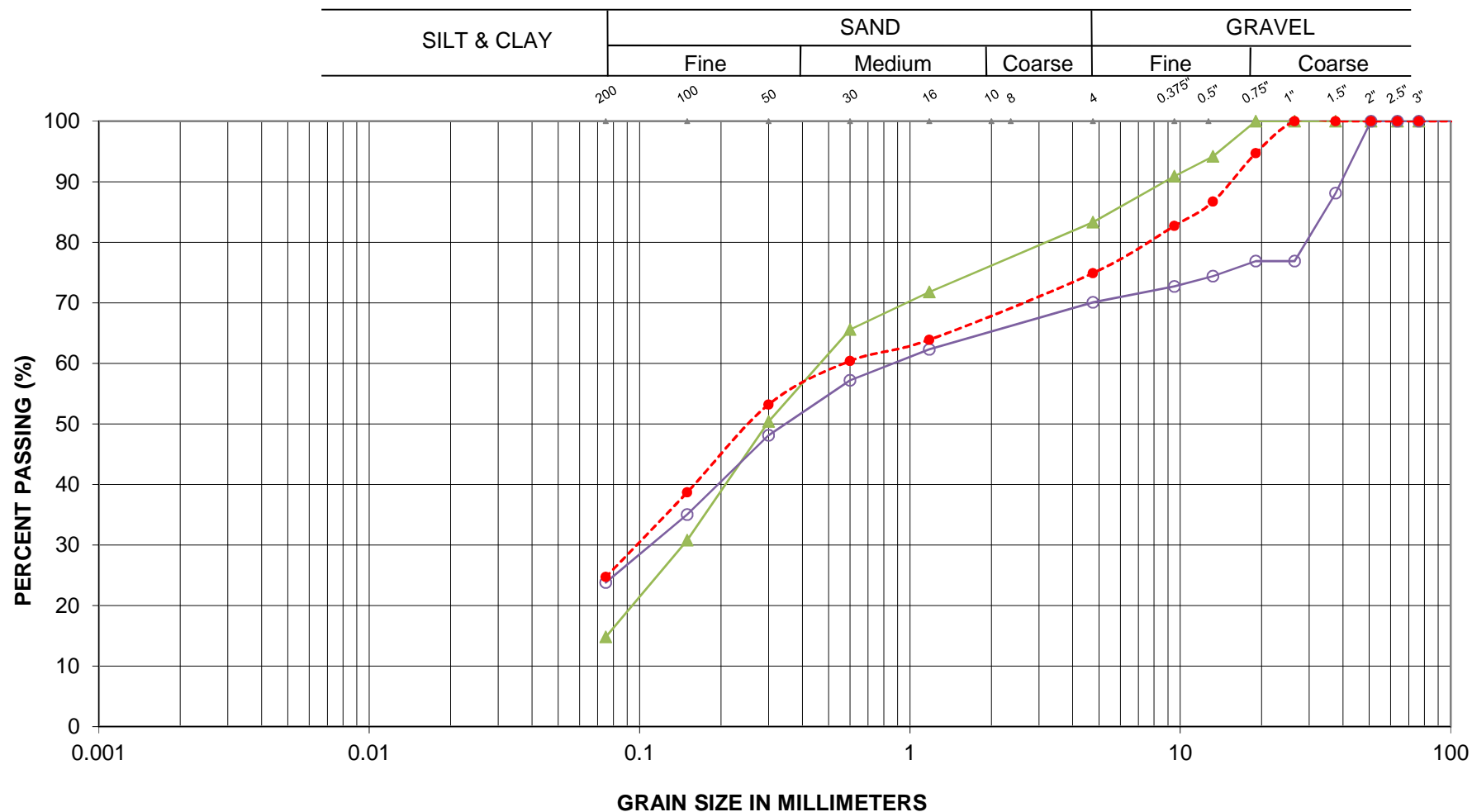


SAND FILL

LOCATION: Hwy 60 Sta. 11+540 Culvert
TWP Sinclair, Ontario

LVM-Merlex, a Division EnGlobe Corp.

FIGURE L-1

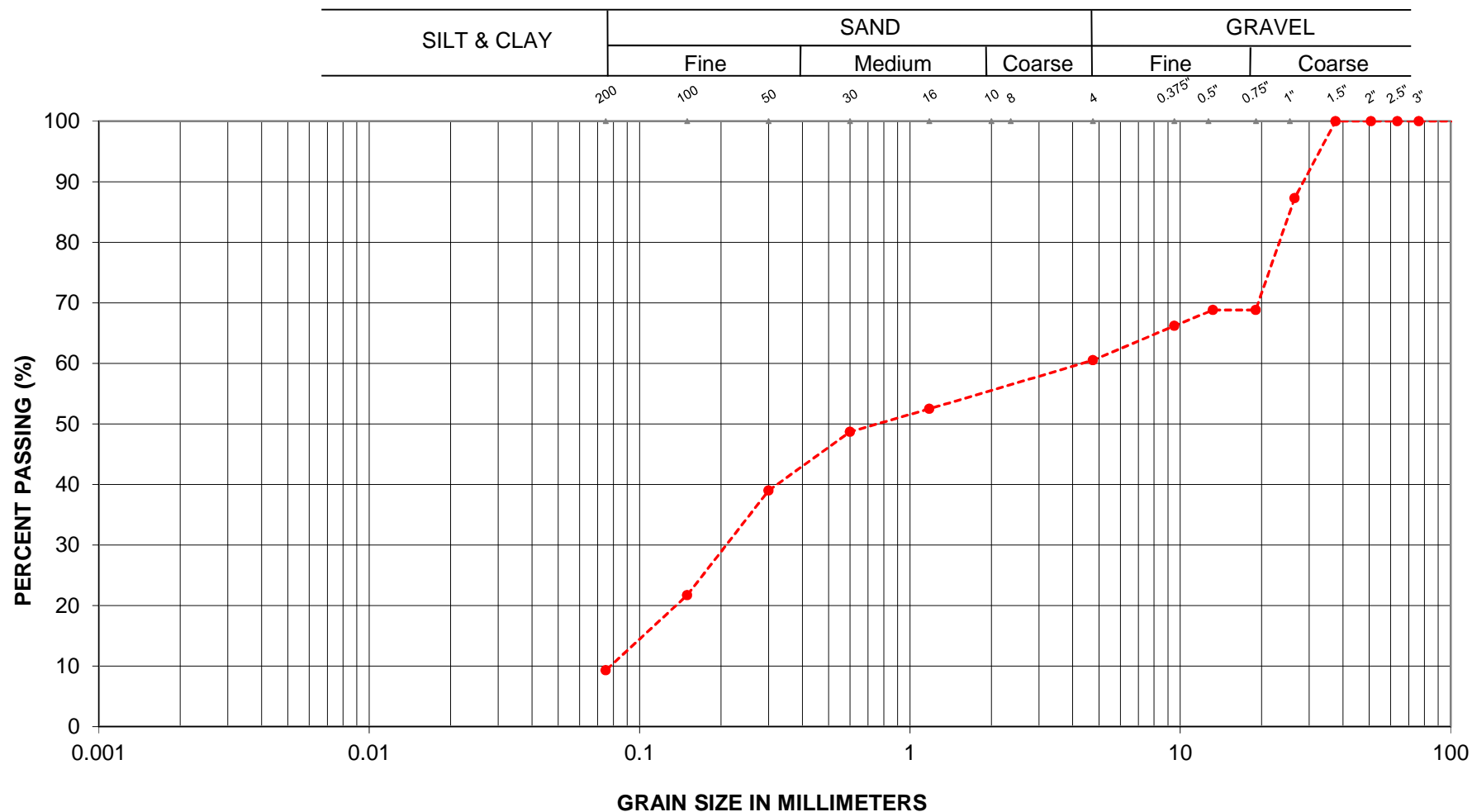
GRAIN SIZE ANALYSIS

SAND

LOCATION: Hwy 60 Sta. 11+540 Culvert
TWP Sinclair, Ontario

LVM-Merlex, a Division EnGlobe Corp.

FIGURE L-2

GRAIN SIZE ANALYSIS

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

SAND and GRAVEL

LOCATION: Hwy 60 Sta. 11+540 Culvert
TWP Sinclair, Ontario

LVM-Merlex, a Division EnGlobe Corp.

FIGURE L-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 (%)				
1	1	0.3					48.5				1			
	2	1.0					32.2				11			
	3	1.8	17	68	15		14.4				38			
2	1	0.3	30	46	24		17.5				3			
	2	1.0	40	51	9		17.7				48			
	3	1.8					17.2				47			
3	1	0.3					5.1				20			
	2	1.1	26	62	12		9.0				21			
	3	1.8					7.6				24			
	4	2.6					11.0				35			
	5a	3.3	9	73	17		12.2				35			
	5b	3.6					3.4							
	6	4.9					13.7				20			
	7	5.6	16	58	19	7	11.4				18			
	8	6.4					11.5				12			
	9	7.2					12.7				11			
	10a	7.8	1	77	22		10.3				16			
	10b	8.0					16.7							
	11	8.7					14.1				5			
	12	9.5					20.0				5			
	13a	10.1					8.0				66			
	13b	10.4					8.7							
	14	10.98	25	50	25		9.22				31			
	15	12.5					16.42				61			

Appendix 4 Photo Essay

Enclosure No. 5:

Photo Essay

Embankment at Culvert Location – Looking East

Photo: 1



Culvert Outlet Location – Looking East

Photo: 2



Project: Hwy 60 – Culvert 11+540

Photos Provided By: LVM

Date: August 2014

Culvert Outlet Location – Looking East

Photo: 3



Culvert Outlet – Looking North

Photo: 4



Project: Hwy 60 – Culvert 13+035

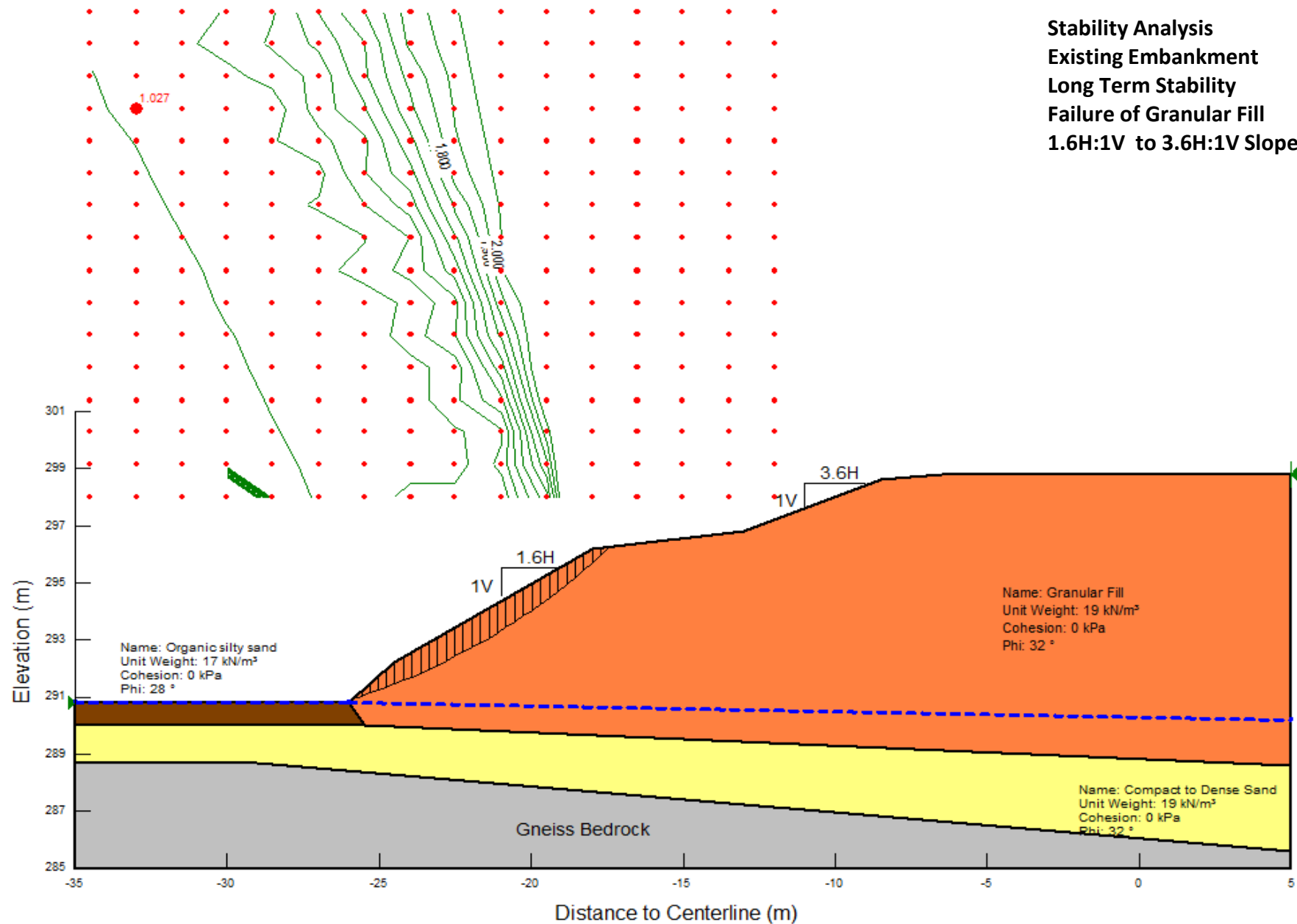
Photos Provided By: LVM

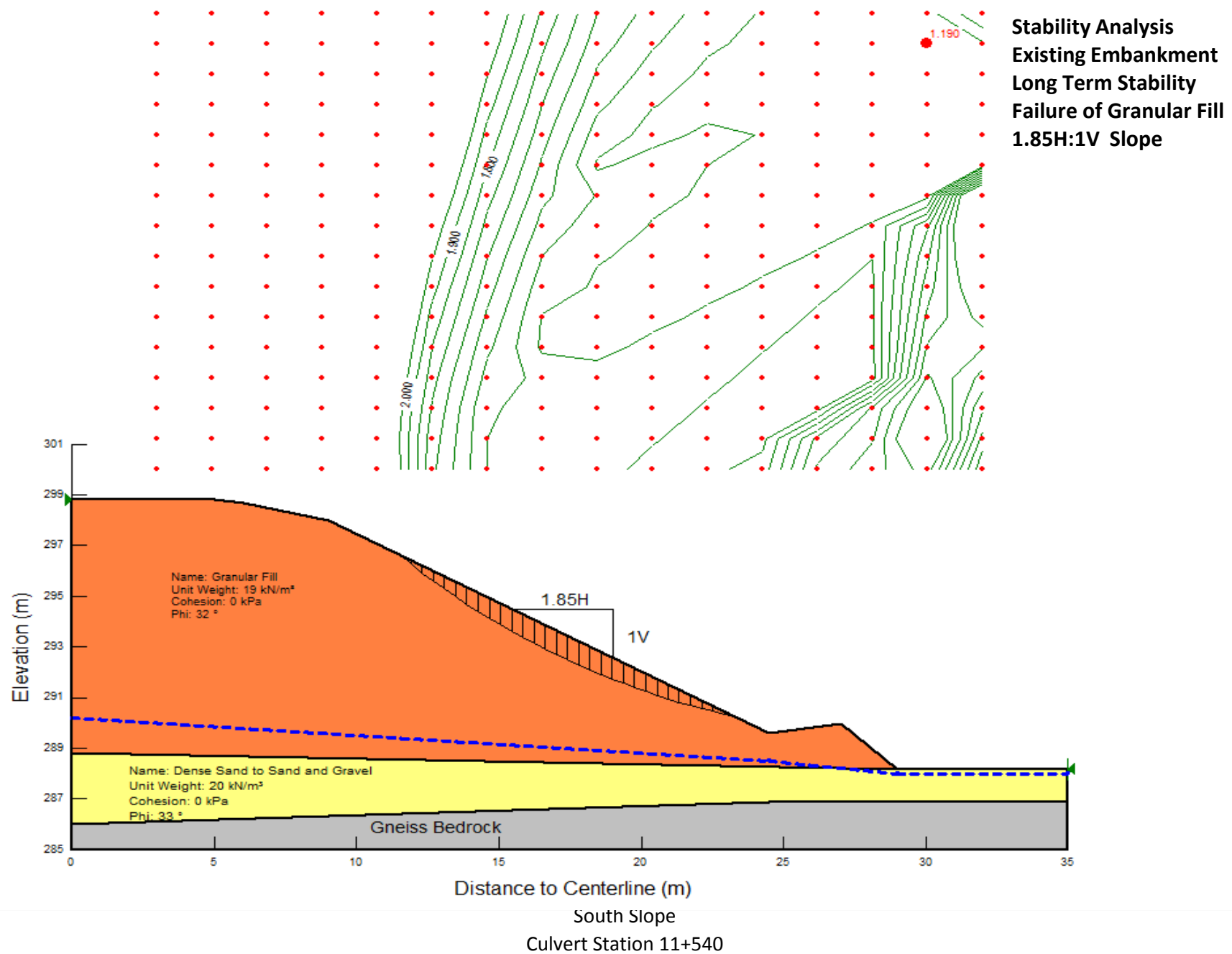
Date: August 2014

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
Table B:	Comparison of Trenchless Techniques

Stability Analysis
Existing Embankment
Long Term Stability
Failure of Granular Fill
1.6H:1V to 3.6H:1V Slopes

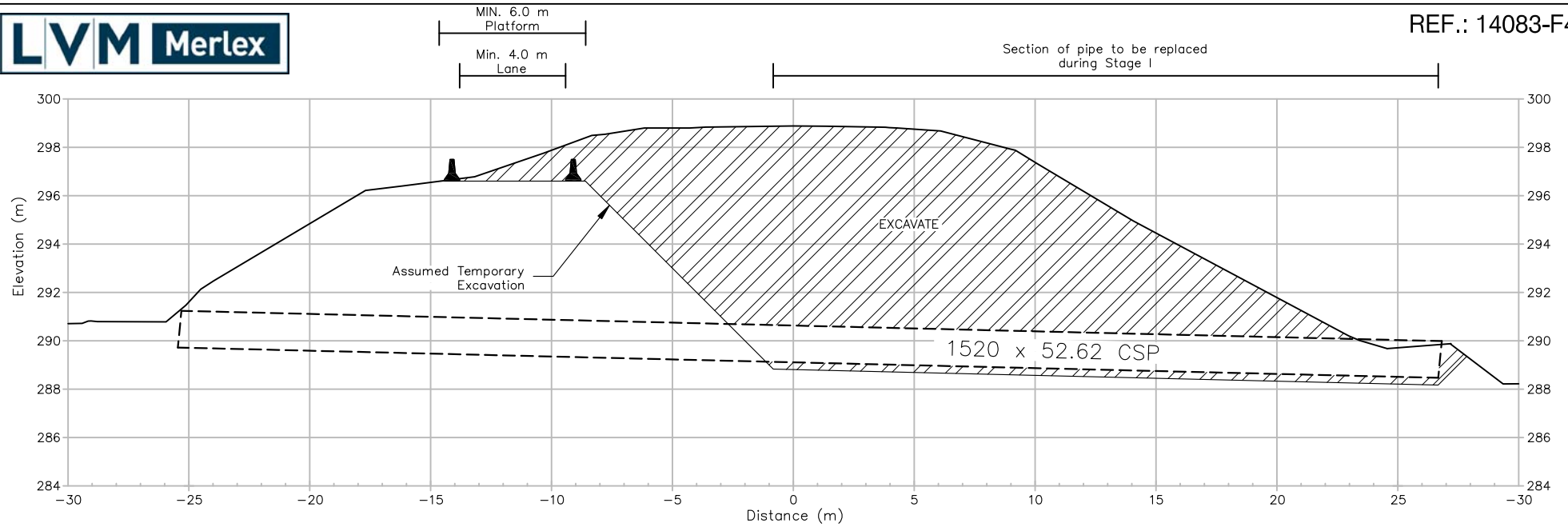




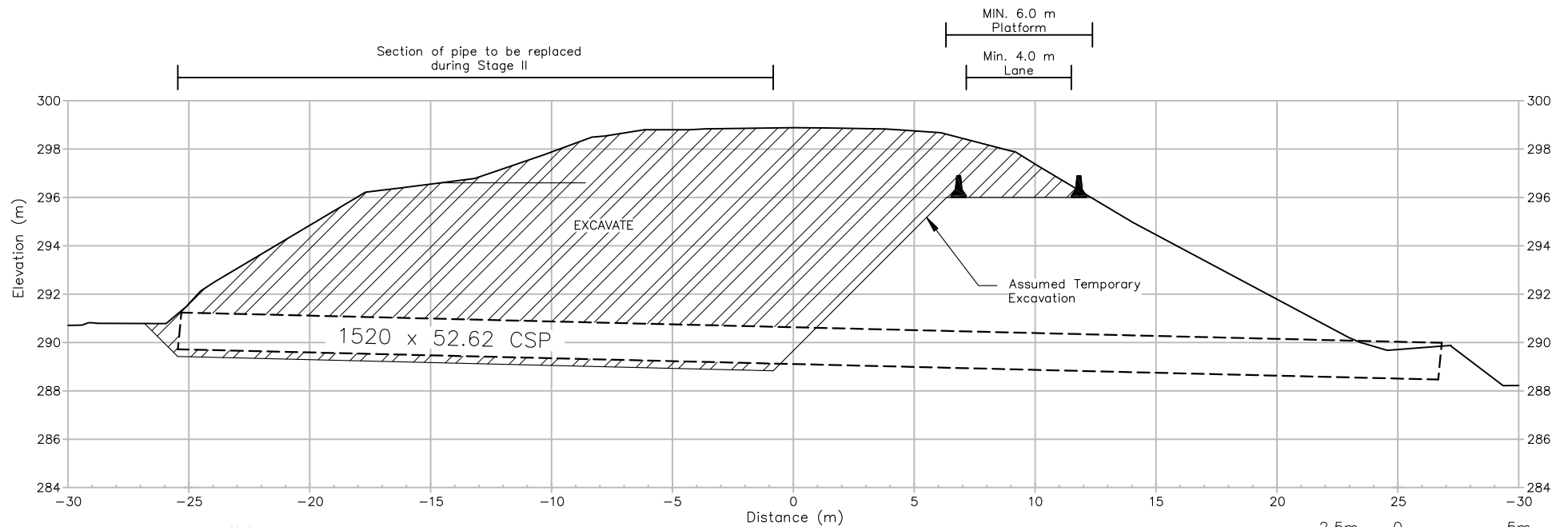
Date: April, 2015

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	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 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	\$ 900/m ²



STAGE I



STAGE II

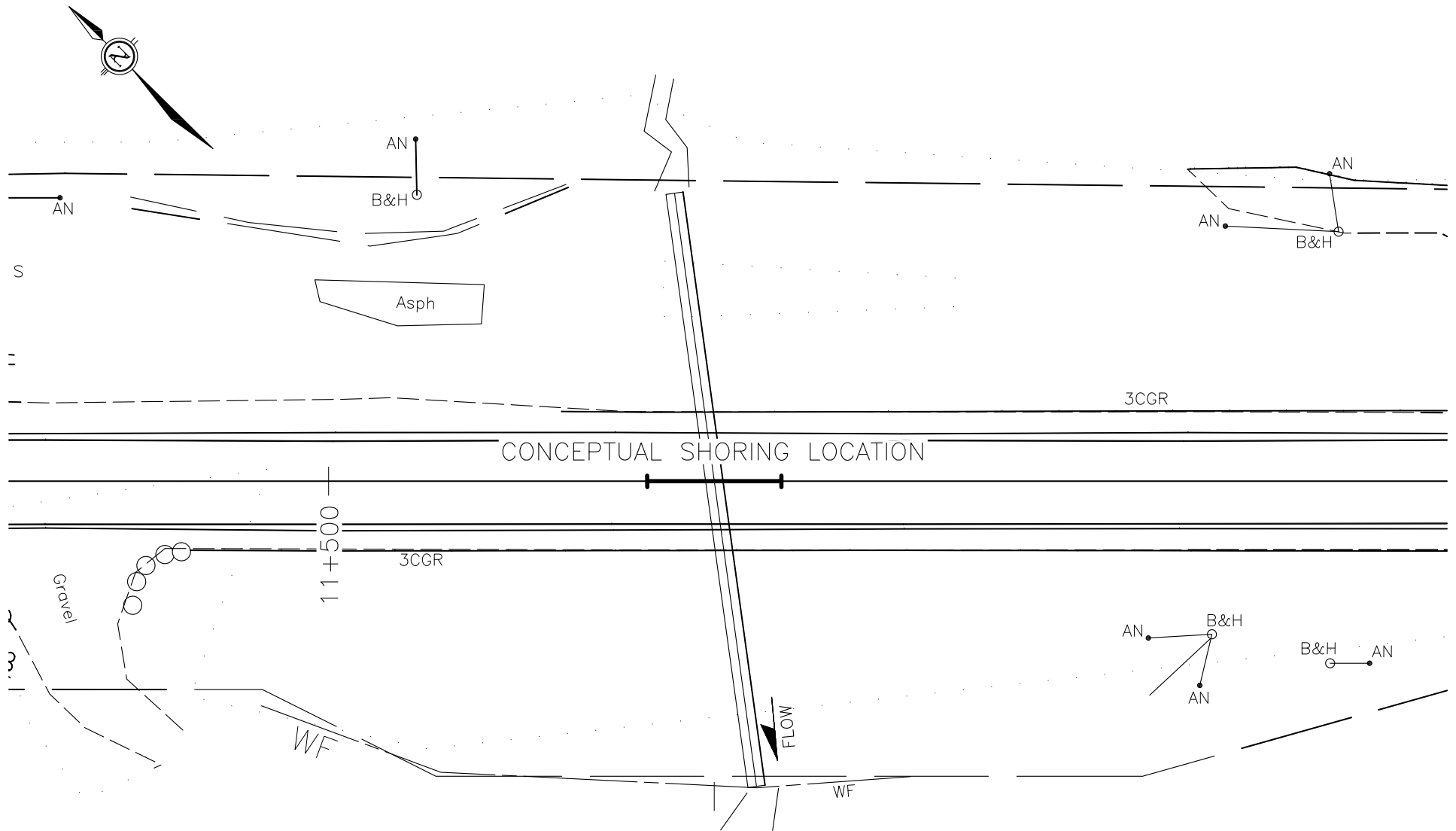
METRIC

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

Note:

Geometry of temporary cut slopes based on assumption of adequate groundwater control carried out by the Contractor during excavation.



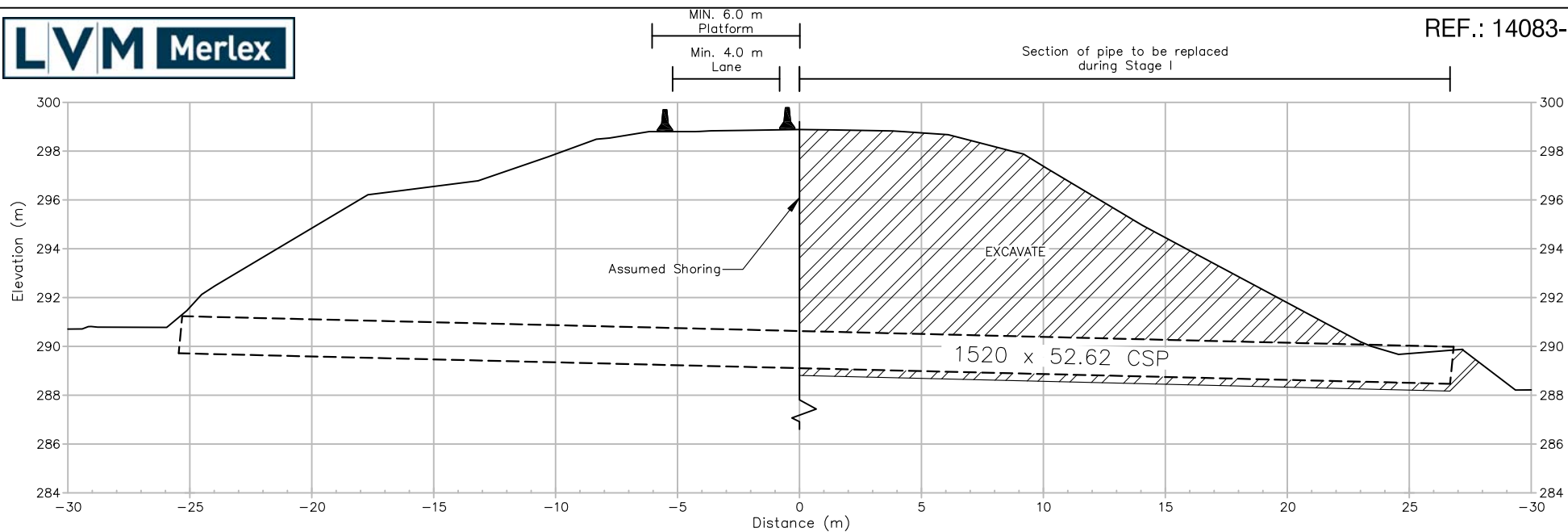


METRIC

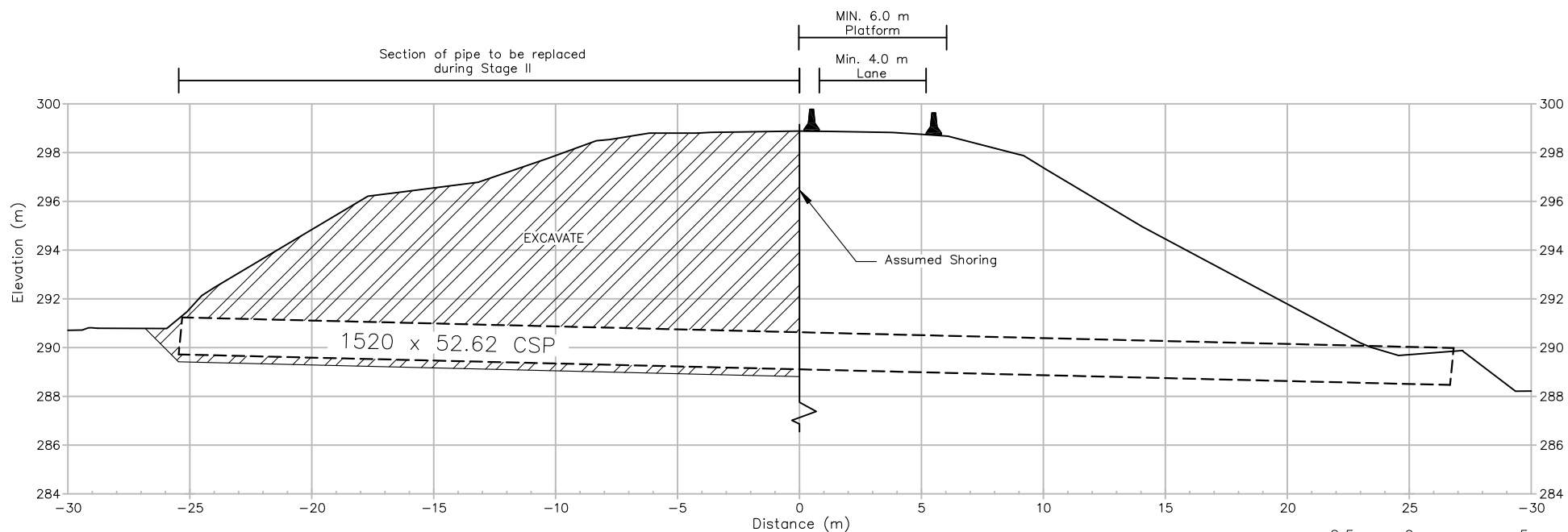
Dimensions are in meters and/or millimeters unless otherwise shown. Stations are in kilometers + meters.

Highway 60, Township of Sinclair - Culvert at Station 11+534
Conceptual Shoring Location Plan

FIGURE SK-4



STAGE I



STAGE II

METRIC

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



Highway 60, Township of Sinclair - Culvert at Station 11+534
Conceptual Shoring Cross-Section

FIGURE SK-5

Table B – Comparison of Trenchless Techniques

Method	Advantages	Disadvantages
Horizontal Direction Drilling	<ul style="list-style-type: none"> Can be used in most ground condition Generally does not require staging pits therefore minimal ground water control required Alignment can be adjusted to avoid obstructions 	Site grades may require longer bore or staging pits
Symmetrix Drilling	<ul style="list-style-type: none"> Can be advanced through bedrock, and most overburden types has been used to advance casings through many rock fill in Scandinavian countries 	Larger drilling equipment may be required
Jack and Bore	<ul style="list-style-type: none"> Good contractor availability Good for shorter tunnel length (<100 m) Good gradient control 	Requires drilling fluid to maintain the bore, which could result in heave
Micro-Tunneling/Pipe Jacking	<ul style="list-style-type: none"> Shield face can accommodate high groundwater conditions Can accommodate cobble/boulders with appropriate shield Can advance boring from 3.2 m diameter maintenance hole in urban areas Alignment can be altered during bore 	Size of pipe limited to 140 mm to 1200 mm
Pipe Ramming	<ul style="list-style-type: none"> Minimal groundwater control required along the installation route (unless required to remove obstruction/old pipe) Can penetrate soils containing cobbles/boulders if obstruction less than casing diameter Has been used successfully in the USA to penetrate a railway rock fill embankment 	Size limited to 140 mm to 1200 mm

