

**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 20+566 - Twp. of Chaffey
GWP 5333-11-00**

FINAL PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT

Date: May 14, 2015
Ref. N^o: 14/07/14083-F3

Geocres No. 31E-346

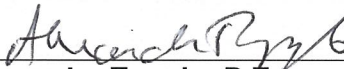


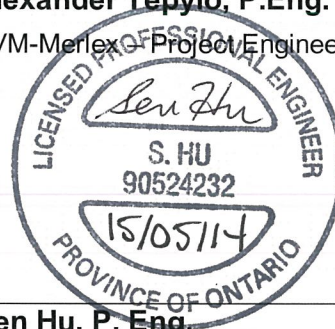
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Final Preliminary Foundation Investigation and Design Report

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REVISION AND PUBLICATION REGISTER		
Revision N°	Date	Modification And/Or Publication Details
00	2015-02-10	DRAFT FIDR Issued
01	2015-05-14	Final FIDR Issued

REPORT DISTRIBUTION	
2 hard copies	AECOM
5 hard copies and 1 electronic copy	MTO Project Manager
1 hard copy and 1 electronic copy	MTO Pavement and Foundations Section, Foundation Group
1 hard copy	File

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 for preparation of a Design-Build Contract. The site is located at Station 20+566 in the Township of Chaffey on Highway 60, some 10.5 km east of Highway 11.

The foundation investigation location was specified by the MTO in the Terms of Reference for work under Agreement No. 5013-E-0032. The terms of reference for the scope of work are outlined in LVM-Merlex's Proposal P-13-051, 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

A Corrugated Steel Pipe (CSP) culvert is located on Highway 60 at Station 20+566 in the Township of Chaffey. The topography in the area of this site is generally rolling. 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 on a granular and rock fill embankment some 4.8 m in height (at centreline), with centerline elevation of 330.4 m at the culvert location. The existing embankment slopes, in the area of the culvert, have been generally established between angles of approximately 1.8H:1V to 1.9H:1V. The culvert at this location is a 910 mm diameter Corrugated Steel Pipe (CSP) culvert, some 30 m in length. Flow through the culvert is from north to south (left to right).

The culvert at this location appears to have failed and rock fill pieces were observed in the culvert up from the outlet end (see Photo 4, Photo Essay, Appendix 4).

Infrastructure at the culvert location consists of overhead wires to the left (north) side of the highway embankment.

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 consists primarily of sands overlying bedrock.

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

3 INVESTIGATION PROCEDURES

The fieldwork for this investigation was carried out during the period of August 13th to September 16th, 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 (north) and outlet (south) 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 boreholes 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(s) through the embankment, the upper portion of the hole, where necessary, was backfilled with an asphalt cold patch to seal the existing asphalt surface.

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

The location of the individual boreholes were 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 20+566, TWP OF CHAFFEY

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 outlet, Borehole No. 2 advanced at the culvert inlet, and Borehole No. 3 advanced through the embankment. At the time of the subsurface investigation, the ground surface elevations at Boreholes Nos. 1 to 3 were recorded at elevations 319.9, 328.5, and 330.2 m, respectively.

4.1.1 Pavement Structure

Borehole No. 3 was advanced through the embankment where a pavement structure consisting of 100 mm asphalt and 300 mm crushed gravel was penetrated.

4.1.2 Embankment Fill

Underlying the pavement structure at Borehole Nos. 3, a layer of fill consisting of brown sand some to with gravel some silt, mixed with rock fill, was penetrated. The natural moisture content measured on samples of this deposit was in the order of 3 to 7%. Gradation analyses were carried out on two (2) samples of this deposit, the results of which indicated 19 to 22% gravel size particles, 62 to 67% sand size particles, and 15 to 17% silt and clay size particles (Figure No. L-1, Appendix 3). Based on SPT 'N' values of 4 to 34 blows per 300 mm penetration, the compactness of this deposit was described as loose to dense. This deposit was encountered to a depth of 2.9 m below grade at Borehole No. 3 (elevation 327.3 m).

4.1.3 Organic Soils

Underlying the embankment fill at Borehole No. 3, and at surface at Borehole Nos. 1 and 2, a layer of silty organic soils, some to with sand was penetrated. Cobble size rock pieces were encountered in this layer at Borehole No. 1. The natural moisture content measured on samples of this layer was in the order of 30 to 54%. This organic soil layer was encountered to depths of 0.8, 0.6 and 3.7 m below ground surface at Borehole Nos. 1 to 3, respectively (elevations 319.1, 327.9, and 326.5 m, respectively).

4.1.4 Sands

Underlying the organic soils at Borehole Nos. 2 and 3, a deposit of grey sand with silt some gravel to gravelly was penetrated. Cobble and boulder size rock pieces were encountered at depth ranging from 4.6 to 6.5 m below ground surface in this deposit at Borehole No. 3. The natural moisture content measured on samples of this deposit was in the order of 13 to 23%. A gradation analysis was carried out on one (1) sample of this deposit, the results of which indicated 11% gravel size particles, 62% sand size particles, and 27% silt and clay size particles (Figure No. L-2, Appendix 3). Based on SPT 'N' values of 40 to 58 blows per 300 mm penetration, this deposit was described as dense to very dense. This deposit was encountered to depths of 2.0 and 6.5 m below grade at Borehole Nos. 2 and 3, respectively (elevations 326.5 and 323.8 m, respectively).

4.1.5 Bedrock

Underlying the above described organic soils at Borehole No. 1 and sands at Borehole Nos. 2 and 3, bedrock was proven by diamond core drilling. The bedrock was described as pink to grey gneiss bedrock. Based on RQD values of 59 to 98% the bedrock was described as fair to excellent quality. Sampling in the bedrock was terminated at depths of 4.5, 5.0, and 9.5 m below grade at Borehole Nos. 1 to 3, respectively (elevations 315.4, 323.5, and 320.7 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 (September 16, 2014), a slight flow was observed through the culvert.

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. 2 to obtain post borehole completion water levels. These levels are recorded on the individual Record of Borehole Log Sheets (Appendix B).

The water levels were measured at elevations 319.7 and 328.5 m at Borehole Nos. 1 and 2, respectively. Groundwater was not encountered within the depth of cave (elevation 328.9 m) at Borehole No. 3.

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 20+566, in the Township of Chaffey, is a 910 mm diameter CSP culvert some 30 m long. The existing culvert invert at centerline is at a depth of some 4.5 m (elevation 325.9 m). The existing highway embankment currently supports two undivided lanes of highway, running in an east-west direction. Flow through the culvert is from left to right (north to south). 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 over granular fills mixed with rock fill. The native material, underlying the embankment fill, generally consisted of dense sands overlying the bedrock at relatively shallow depths.

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 present below the existing embankment are considered adequate for support of a culvert and for a conventional highway embankment of this height. Bearing resistance should not be a major issue provided the natural bearing surface is not disturbed during construction and groundwater is controlled throughout construction, as discussed in Section 5.6.

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 475 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

A stability analysis, using the GEO-SLOPE computer program, Slope/W (GeoStudio 2007, version 7.17, Geo-Slope International Ltd.), was carried out at this location with standard

embankment slopes of 2.0H:1.0V in granular fill. For the purposes of these analyses, the materials were modeled using the following parameters;

PARAMETER	MATERIAL		
	EMBANKMENT FILL	ORGANIC SOILS	SANDS
Unit Weight (kN/m ³)	19	10	18.5
Effective Friction Angle (degrees)	35	10	32

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

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 at Borehole No. 3 consisted of dense to very dense sands. A review of the condition of the pavement surface, at the culvert locations, revealed some 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 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 19 mm clear stone bedding should be used, which would aid in dewatering operations. During backfilling, the embedment fill should be placed in 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 in accordance with OPSS 501.

A precast concrete rigid frame box culvert can also be considered for culvert replacement at this site. However, in consideration of the existing culvert size (910 mm), a rigid frame box culvert will likely not be used.

The inlet and outlet stream bed shall be protected with a rip-rap (R-50 size as per OPSS 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/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 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 fill should be placed in 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 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 STAGING CONSIDERATIONS

The invert elevation of the existing culvert is at 325.9 m, with the top of the embankment at elevation 330.4 m at centerline. As such, the embankment at this location is some 4.5 m in height above the culvert invert at the centerline. Therefore, a minimum 4.8 m deep excavation (i.e. to elevation 325.6 m) will be required, at centreline, in consideration of a 300 mm thick layer of bedding/embedment material. Excavations to greater depths (i.e. to elevation 323.2 m) will be required to the right (outlet) end of the culvert, considering the slope of the culvert. The present platform width at this location is some 13 m as can be seen on the cross section on Drawing No. 2. The platform width at this location, as is, will not be sufficient to carry out an open excavation 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 1.4 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.

5.4.1 Staged Construction

As noted, the platform at this location, as is, is of insufficient width to carry out an open excavation using staged construction, 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 328.6 m.
- Limit traffic to a single lane on the left, with a minimum platform width of 6 m, under 24/7 traffic control.
- Open cut excavate, to the right, and install approximately 10 m of new culvert.
- Reconstruct the embankment on the right, with a minimum platform width of 6 m for traffic.
- Divert the single lane of traffic to the right and continue open excavation to install the remainder of the culvert on the left.
- As the width of the platform increases on the right, the vertical alignment can be raised, 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 4.8 m of granular fills mixed with rock fill. The embankment fill is generally underlain by dense sands. As noted rock fill was encountered in the embankment. Considering the presence of rock fill in the embankment, advancing a temporary retaining system (i.e. driven sheet piles) through the rock fill may be problematic. The presence and quantity of rock fill in the embankment fill will be determined when additional boreholes are put down during detailed design. Several approaches to constructing a protection system are described in the following. See Table A, Appendix 5, for advantages and disadvantages for the different type of protection system considered for this site. Conceptual shoring locations are illustrated on Figure No. SK-4, Appendix 5.

One method to construct a protection system would be to penetrate the rock fill in the embankment with H piles (soldier piles) extending into the underlying sands and/or onto bedrock and install lagging. Pre-drilling may likely be required to advance the H piles through the rock fill and into the underlying the bedrock to allow sufficient toe resistance. The H piles would be installed at an interval of 2.5 to 3 m apart and the lagging would be installed as the

excavation progresses. A waler and raker system or tie back anchor system would have to be installed as the excavation advances. The contractor must be prepared to address large pieces of rock fill and control groundwater as the excavation progresses, without compromising the adjacent active lane of traffic.

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

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

A_s = effective unit surface area of the anchor

L_s = effective embedment length of the anchor

α_g = anchorage coefficient use 1.0 for granular backfill

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

Alternatively, a caisson wall or drilled micropile system with an intermediate support system of reinforced shotcrete, to act as lagging, could be considered for roadway protection at this site. One method of constructing this system would be to drill in micropiles, advancing to either side of the culvert below the invert several meters into the compact to dense sands or probably into bedrock, depending upon the size and capacity of the micropiles. Over the actual culvert location, the piles would be carried down to top of culvert grade followed by bracing, with a suitably sized waler and anchorage system, tied into the full depth piling at the culvert sides, in order to provide support at the top of the piling over the culvert barrel. Depending on the section properties of the retaining structure, walers and bracing struts or ground anchor support systems will probably be required. As the excavation progresses downward in 1 to 1.2 m lifts, a reinforced shotcrete, tied into the piles, is applied. Once one half of the box culvert construction is complete, a system of buried anchors could be installed to tie back the micropiles as the highway fill is brought up to grade. When the excavation on the opposite side reaches the anchor depths, a support waler, if required, can be placed and tensioned to support the shotcrete as specified in the contractor's approved shoring design. However these shoring system are generally more costly, as such are not recommended at this site.

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, Appendix 5. Conceptual shoring locations are illustrated on Figure No. SK-4, Appendix 5.

The protection system can be designed using the lateral earth pressure parameters as outlines in Section 5.5.

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

K_a = active earth pressure,

γ = unit weight, and

H = height of wall above the base of excavation.

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

5.5 LATERAL EARTH PRESSURES

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

PARAMETER	GRANULAR A	GRANULAR B TYPE I	EMBANKMENT FILL	SANDS
Unit Weight (kN/m^3)	22.8	21.2	19	18.5
Angle of Internal Friction	34°	31°	35°	32°
Coefficient of Active Earth Pressure (K_a)	0.28	0.32	0.27	0.31
Coefficient of Passive Earth Pressure (K_p)	3.54	3.12	3.69	3.25
Coefficient of Earth Pressure at Rest (K_o)	0.44	0.48	0.47	0.47

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.

5.6 EXCAVATION, DEWATERING, AND EMBANKMENT RECONSTRUCTION

All excavations greater than 1.2 m in depth must, at a minimum, be sloped or shored in accordance with the Occupational Health and Safety Act Regulations for Construction Projects. The embankment material, above the water table, is considered a Type 3 soil as defined in the Occupational Health and Safety Act and Regulations for Construction Projects. Temporary

open excavations above the groundwater table, could be cut back at an angle of 1H:1V, provided they are monitored continuously, however, below the groundwater table, the side slopes will have to be cut back to an angle of 2H:1V, possibly shallower, dependent upon the Contractors' chosen method of controlling the groundwater. Temporary open cuts with a slope of 1H:1V cannot be left unattended (i.e. overnight, during breakdowns, etc.). If work must stop for extended periods of time, the temporary slopes must be flattened to a minimum angle of 2H:1V.

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

Bedrock was not encountered at the borehole locations within the anticipated depth of excavation, therefore bedrock excavation and/or blasting operations are not anticipated. However, the bedrock surface in the area can be highly variable over relatively short horizontal distances. 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 328.8 m at the culvert invert at the time of this investigation and excavations to an approximate elevation 325.6 m (at centreline) will be required to install the culvert and bedding. As such dewatering will be required during excavation and culvert installation.

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. This method of groundwater control is generally only effective when the groundwater in the excavation is less than a depth of some 1 m above the final base of the excavation. To effectively lower the groundwater to a greater depth, a more sophisticated groundwater control system, such as a well points or closed sheeting, would have to be considered. To provide a stable working surface the water level must be controlled to below the base of excavation. When wet, silty/sandy subgrades can become easily disturbed, and can lose a significant portion of its natural bearing capacity.

A cofferdam, constructed of earth fill, sand bags, or water filled bag (i.e. aquadam) can be considered at this site. Steel sheet piles may also be considered for controlling stream flow, however, the generally shallow bedrock at the inlet and outlet (0.8 and 2.0 m at Borehole Nos. 1 and 2, respectively), may limit the penetration of a steel sheet pile type cofferdam. For base design, sheet piles should extend a minimum depth below base of proposed excavation equal to the height of water above the base of excavation. By-pass pumping can be carried out to divert the stream flow at the time of construction. It is recommended that by-pass pumping,

through a temporary culvert installed through the embankment, be carried out to divert the stream flow past the work area isolated with the cofferdam system.

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.

5.7 CONSTRUCTION CONCERNS

Considering the nature of the granular fill embankment, which is mixed with rock fill, issues with driving closed sheeting may develop. 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 detailed design.

6 STATEMENT OF LIMITATIONS

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

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

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

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

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

Appendix 1 Key Plan

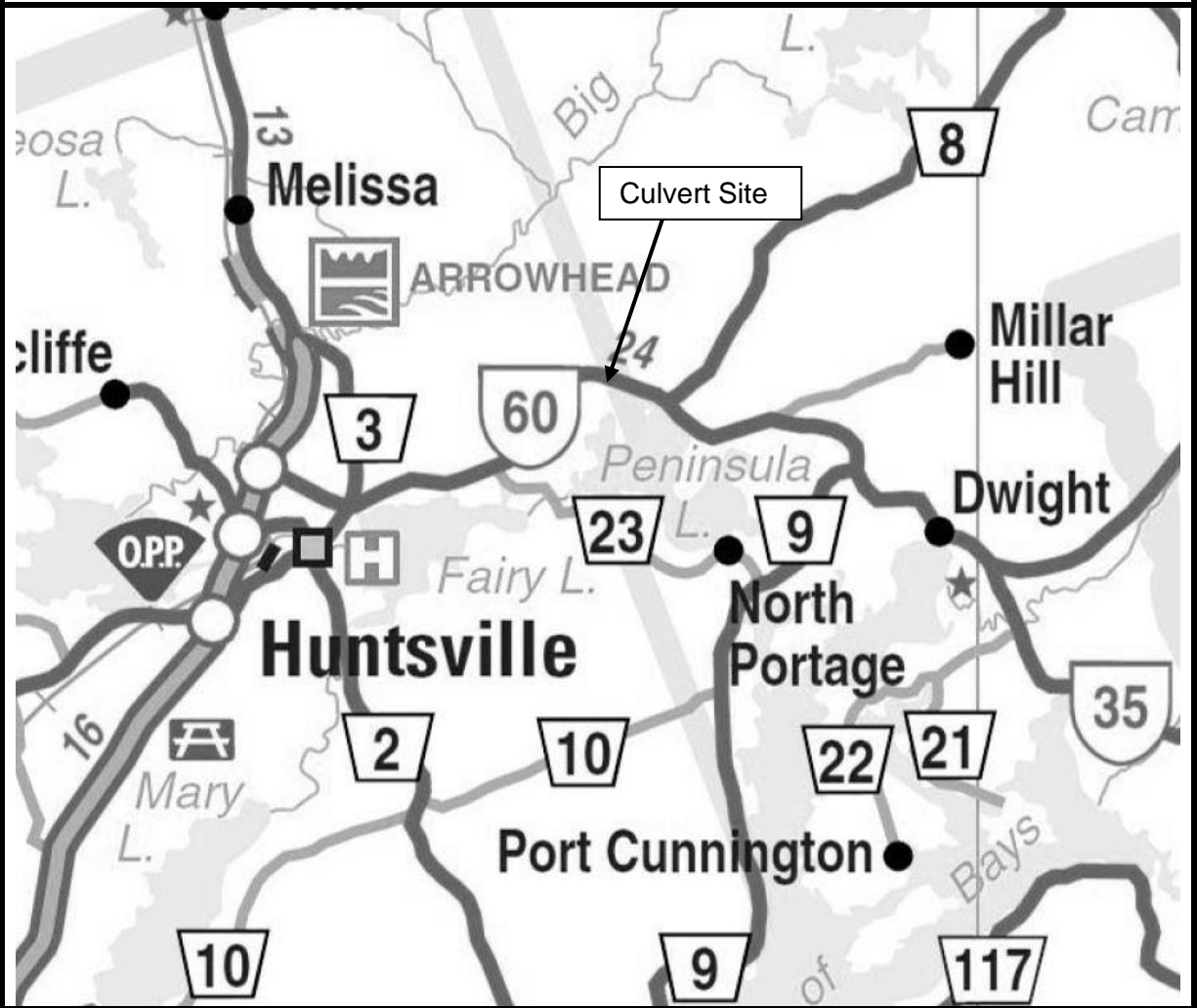
Drawing No. 1

Key Plan

KEY PLAN

Drawing No. 1

NOT TO SCALE



FINAL
FOUNDATION INVESTIGATION
AND DESIGN REPORT
GWP 5333-11-00
Highway 60
Culvert 20+566, Twp of Chaffey



Reference No: 14/07/13083-F3

May 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
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-F3 DATUM Geodetic LOCATION N 5025121.6 E 334582.8 - Chaffey Twp., Station 20+566 ORIGINATED BY JL
 PROJECT GWP 5333-11-00, Highway 60 BOREHOLE TYPE Track Mounted CME 45 - Hollow Stem Augers COMPILED BY SH
 CLIENT AECOM DATE (Started) 13 August 2014 TIME
 DATE (Completed) 14 August 2014 (Completed) 1:00:00 PM CHECKED BY MAM

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100					
319.9	Ground Surface		1	SS	10/0mm												
0.0	ORGANIC SOILS - silty some sand cobble size rock pieces		2	SS	20/0mm												
319.1	BEDROCK - grey/ pink gneiss		3	RC	Rec = 100% RQD = 59%												
0.8	fair to good quality		4	RC	Rec = 100% RQD = 88%												
			5	RC	Rec = 100% RQD = 88%												
315.4	End of Borehole																
4.5																	

WATER LEVEL RECORDS	
Date (dd/mm/yy)/Time	Water Depth (m)
1) 14/8/14 10:45:00 AM	0.25
2)	-
3)	-

COMMENTS

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

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 - F3.GPJ MEL-GEO.GDT 15/5/15

METRIC

RECORD OF BOREHOLE NO. 2



REFERENCE 14/07/14083-F3 DATUM Geodetic LOCATION N 5025157.0 E 334596.8 - Chaffey Twp., Station 20+567 ORIGINATED BY JL
 PROJECT GWP 5333-11-00, Highway 60 BOREHOLE TYPE Track Mounted CME 45 - Hollow Stem Augers COMPILED BY SH
 CLIENT AECOM DATE (Started) 21 August 2014 TIME
 DATE (Completed) 21 August 2014 (Completed) 1:30:00 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	SHEAR STRENGTH kPa					
328.5	Ground Surface												
0.0	ORGANIC SOILS - silty, with sand		1	SS	12								
327.9	SAND - with silt some gravel		2	SS	50/76mm								
0.6	brown (very dense)		3	SS	58								
326.5	BEDROCK - grey gneiss		4	RC	Rec=98% RQD=98%								
2.0	good to excellent quality		5	RC	Rec=100% RQD=87%								
323.5	End of Borehole												
5.0													

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 (dd/mm/yy)/Time	Water Depth (m)	Cave In (m)
	1) 21/8/14 1:40:00 PM	0	3.96
	2)	-	-
	3)	-	-

MEL-GEO 14083 - BOREHOLE LOGS - F3.GPJ MEL-GEO.GDT 15/5/15

METRIC**RECORD OF BOREHOLE NO. 3**

REFERENCE 14/07/14083-F3 DATUM Geodetic LOCATION N 5025149.3 E 334594.0 - Chaffey Twp., Station 20+567 ORIGINATED BY JL
 PROJECT GWP 5333-11-00, Highway 60 BOREHOLE TYPE Truck Mounted CME 45 - Hollow Stem Augers COMPILED BY SH
 CLIENT AECOM DATE (Started) 16 September 2014 TIME 16 September 2014 (Completed) 2:00:00 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			SHEAR STRENGTH kPa							WATER CONTENT (%)											
							20 40 60 80 100	20 40 60 80 100	20 40 60																	
330.2	Ground Surface																									
0.0	100 mm Asphalt 300 mm Crushed Gravel FILL - sand some to with gravel some silt mixed with rock fill brown (loose/dense)		1	SS	34					○				19 67 (15)												
			2	SS	15					○																
			3	SS	12					○																
			4	SS	4					○				22 62 (17)												
327.3	ORGANIC SOILS - silty black		5	SS	2						○															
326.5																										
3.7	SAND - with silt some gravel to gravelly brown (dense/very dense)		6	SS	40						○															
			7	SS	50/25 mm																					
			8	RC	Rec=40% RDR=0%																					
	cobble/boulder size rock pieces below 4.6 m depth switch to casing at 5.0 m depth																									
323.8	BEDROCK - grey gneiss good to excellent quality		9	RC	Rec=100% RDR=91%																					
6.5																										
			10	RC	Rec=100% RDR=84%																					
321																										
320.7	End of Borehole																									
9.5																										
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 (dd/mm/yy)/Time</th> <th>Water Depth (m)</th> <th>Cave In (m)</th> </tr> </thead> <tbody> <tr> <td>1) 16/9/14 9:40:00 AM</td> <td>Dry</td> <td>2.16</td> </tr> <tr> <td>2) 16/9/14 2:00:00 PM</td> <td>Dry</td> <td>1.32</td> </tr> <tr> <td>3)</td> <td>-</td> <td>-</td> </tr> </tbody> </table>							Date (dd/mm/yy)/Time	Water Depth (m)	Cave In (m)	1) 16/9/14 9:40:00 AM	Dry	2.16	2) 16/9/14 2:00:00 PM	Dry	1.32	3)	-	-
Date (dd/mm/yy)/Time	Water Depth (m)	Cave In (m)																								
1) 16/9/14 9:40:00 AM	Dry	2.16																								
2) 16/9/14 2:00:00 PM	Dry	1.32																								
3)	-	-																								

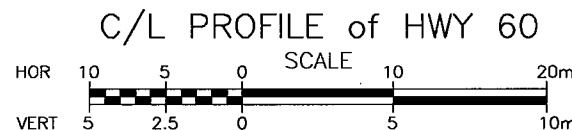
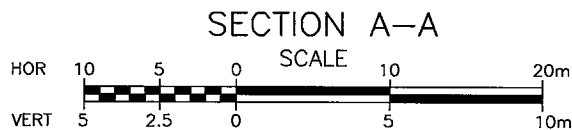
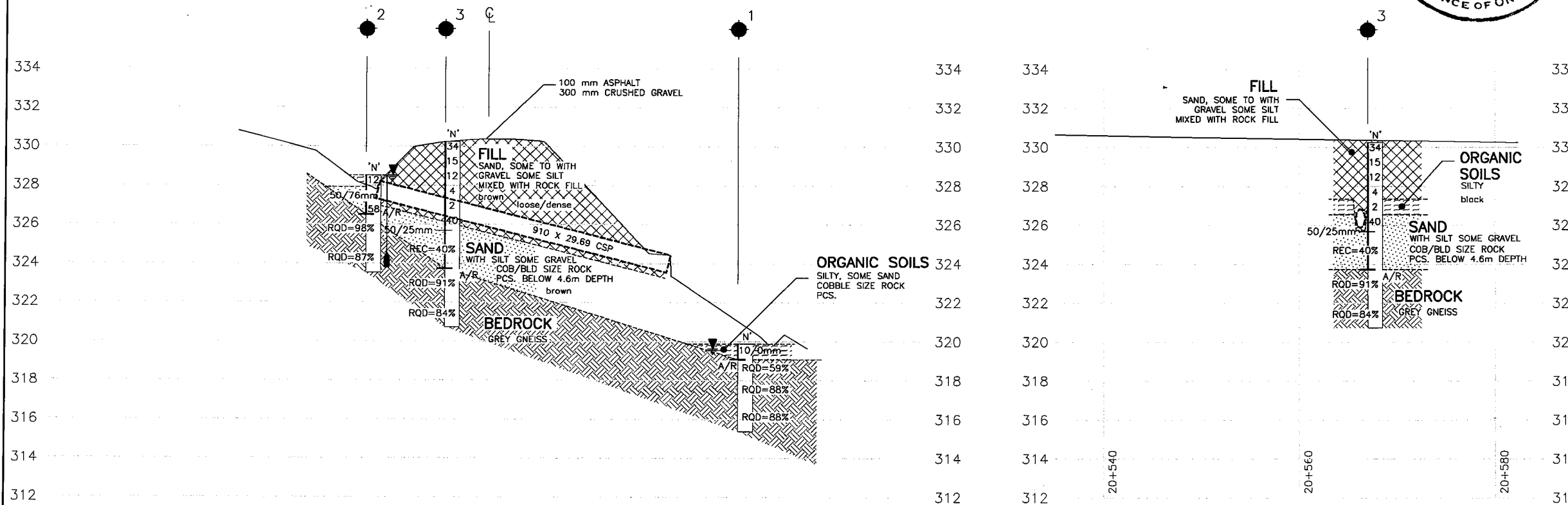
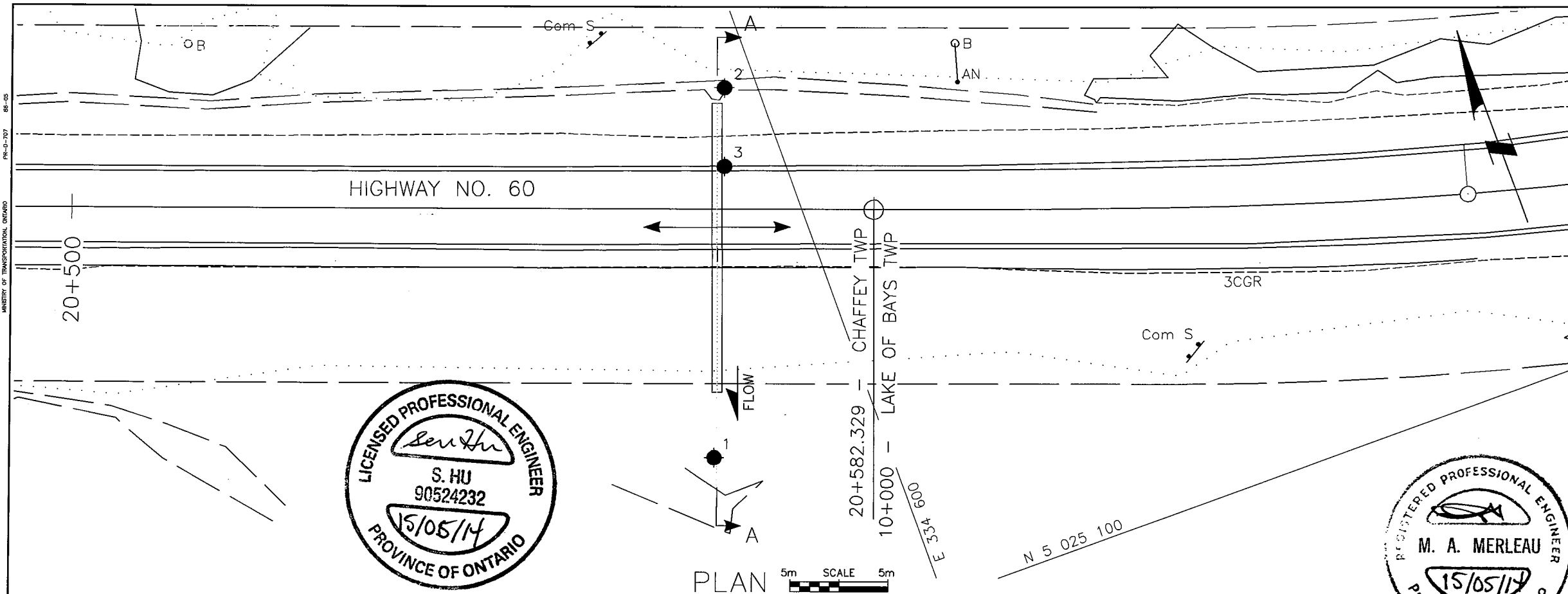
The stratification lines represent approximate boundaries. The transition may be gradual.

MEL-GEO 14083 - BOREHOLE LOGS - F3.GPJ MEL-GEO.GDT 15/5/15

Appendix 3 Borehole Plan and Lab Data

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

CAD FILE LOCATION AND NAME: \\2014\14083 - PAV & FDN, Hwy 60, Huntsville (ACEDON\FOUNDATIONS\Drawings\F2 and F3\F3\Working - Do Not Move or Delete Files\14083-F3 - FINAL - Drawing Plg, Culvert at 20+566.dwg
MODIFIED: 4/9/2015 9:30:30 AM BY: GRASSER
DATE PLOTTED: 5/6/2015 1:53:03 PM BY: RYAN GRASSER



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

DRAWING NOT TO BE SCALED
50mm ON ORIGINAL DRAWING

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

HWY 60
CULVERT AT STATION 20+566
CHAFFEY TOWNSHIP

BOREHOLE LOCATIONS
AND SOIL STRATA

DRAWING

2

LVM Mertex

METRIC

LEGEND

Borehole

Borehole w/ Dynamic Cone Penetration Test

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

Blows/0.3 m (60° Cone, 475 J/blow)

Water Level at Time of Investigation

Auger Refusal at Elevation

End of Sampling

Piezometer

BOREHOLE No.	ELEVATION	O/S	NORTHING	EASTING
1	319.9	25.5m Rt	5025121.6	334582.8
2	328.5	12.5m Lt	5025157.0	334596.8
3	330.2	4.4m Lt	5025149.3	334594.0

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 23, 2014.

GEOCRES No. 31E-346

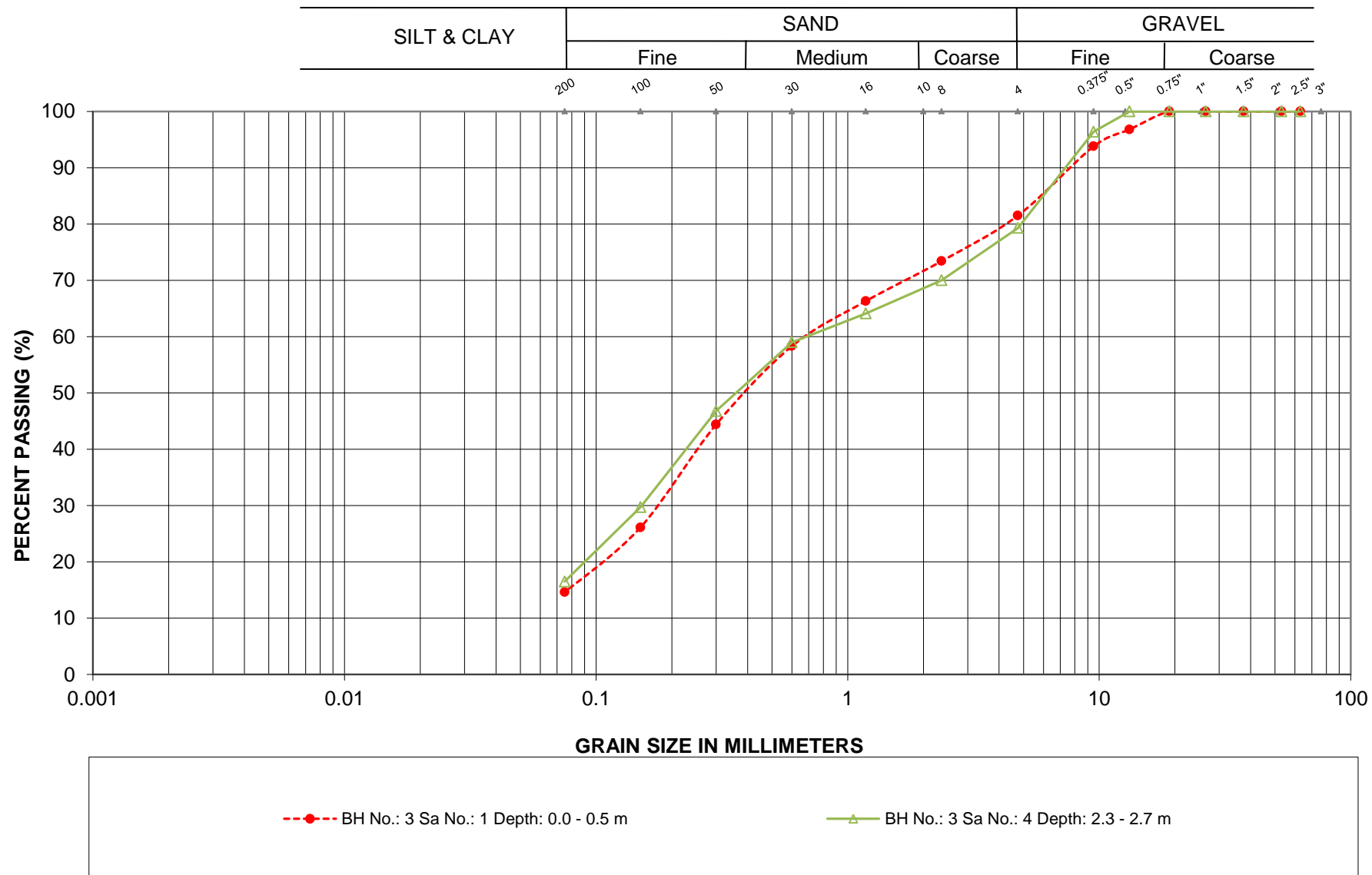
REVISIONS

NO.	DATE	BY	DESCRIPTION
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2	APR/15	RG	FINAL

DESIGN	CHK	CODE	LOAD	DATE
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DESCRIPTION	DATE
STRUCT	APR/15
SCHEME	DWG
DWG	2

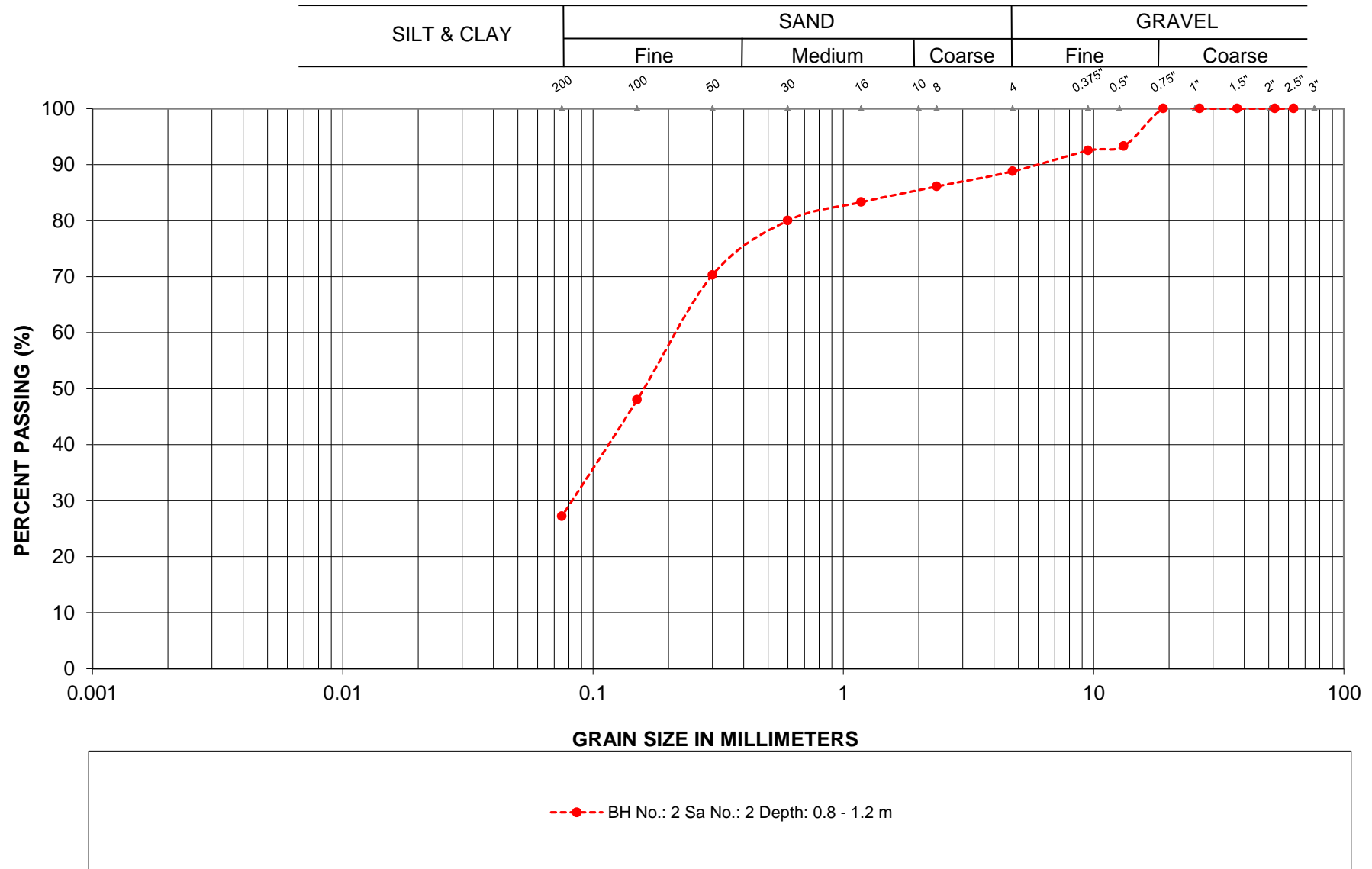
GRAIN SIZE ANALYSIS



LOCATION: Hwy 60 CSP, Station 20+566
Chaffey TWP, Ontario

EMBANKMENT FILL

GRAIN SIZE ANALYSIS



LOCATION: Hwy 60 CSP, Station 20+566
Chaffey TWP, Ontario

SAND

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	0.0					54.5				10/0mm			
	2	0.8									20/0mm			
	3	0.8												Rec=100%, RQD=59%
	4	1.7												Rec=100%, RQD=88%
	5	3.1												Rec=100%, RQD=88%
2	1	0.0					36.7				12			
	2	0.8	11	62	27		16.9				50/76mm			
	3	1.5					13.0				58			
	4	2.0												Rec=98%, RQD=98%
	5	3.5												Rec=100%, RQD=87%
3	1	0.0	19	67	15		5.0				34			
	2	0.8					3.5				15			
	3	1.5					3.0				12			
	4	2.3	22	62	17		8.0				4			
	5	3.1					30.1				2			
	6	3.8					22.8				40			
	7	4.6									50/25 mm			
	8	5.0												Rec=40%, Rec(BDR)=0%
	9	6.5												Rec=100%, RQD=91%
	10	8.0												Rec=100%, RQD=84%

Appendix 4 Photo Essay

Enclosure No. 5:

Photo Essay

Existing Embankment – Looking East

Photo: 1



Rock fill in embankment – South side

Photo: 2



Project: Hwy 60 – Culvert at Station 20+566, Chaffey Township

Photos Provided By: LVM

Date: June 2014

Culvert outlet – Looking South

Photo: 3



Culvert Outlet – Looking north

Photo: 4



Project: Hwy 60 – Culvert at Station 20+566, Chaffey Township

Photos Provided By: LVM

Date: June 2014

Looking through culvert – Looking north

Photo: 5



Culvert Inlet – Looking South

Photo: 6



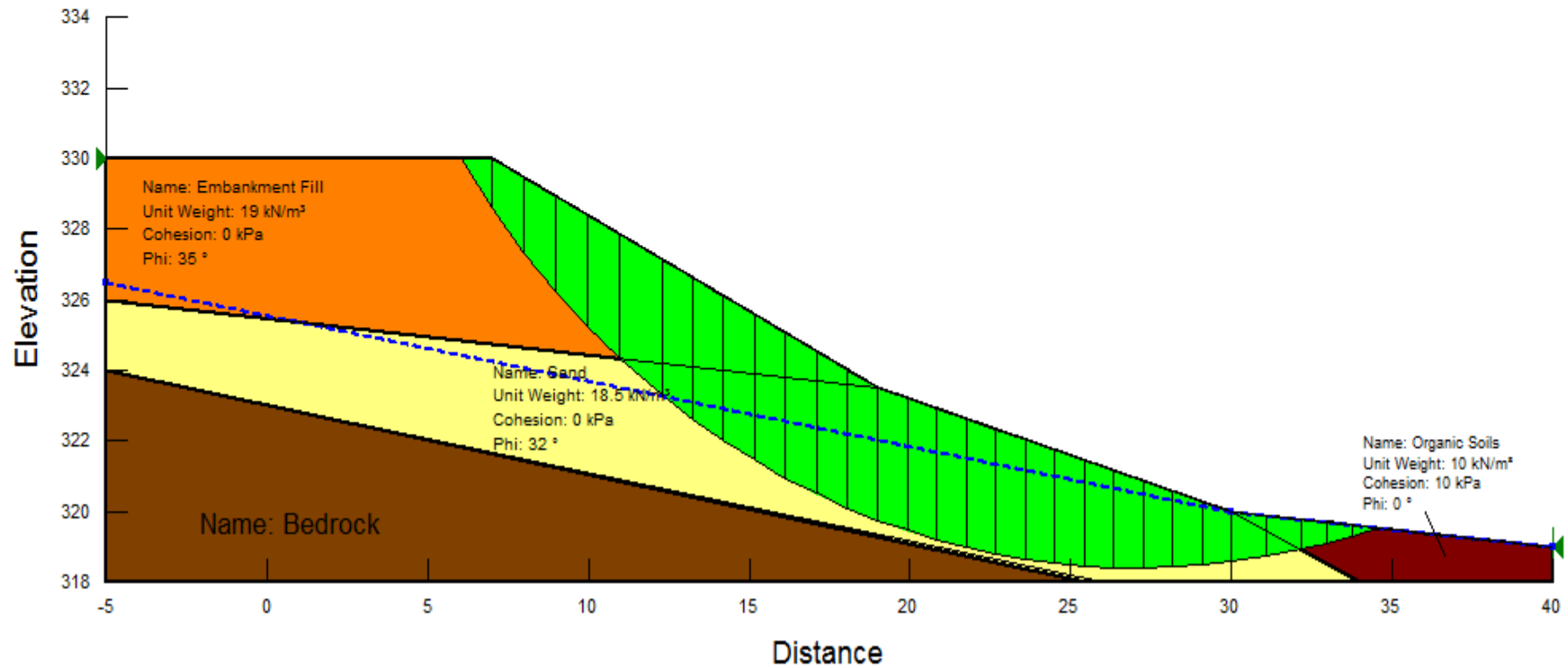
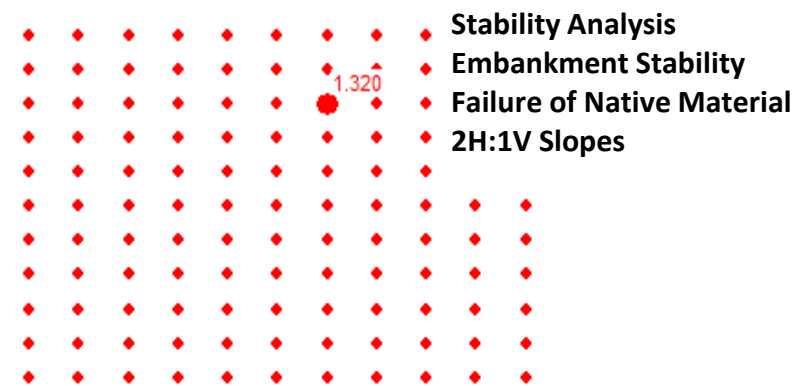
Project: Hwy 60 – Culvert at Station 20+566, Chaffey Township

Photos Provided By: LVM

Date: June/August 2014

Appendix 5 Design Data

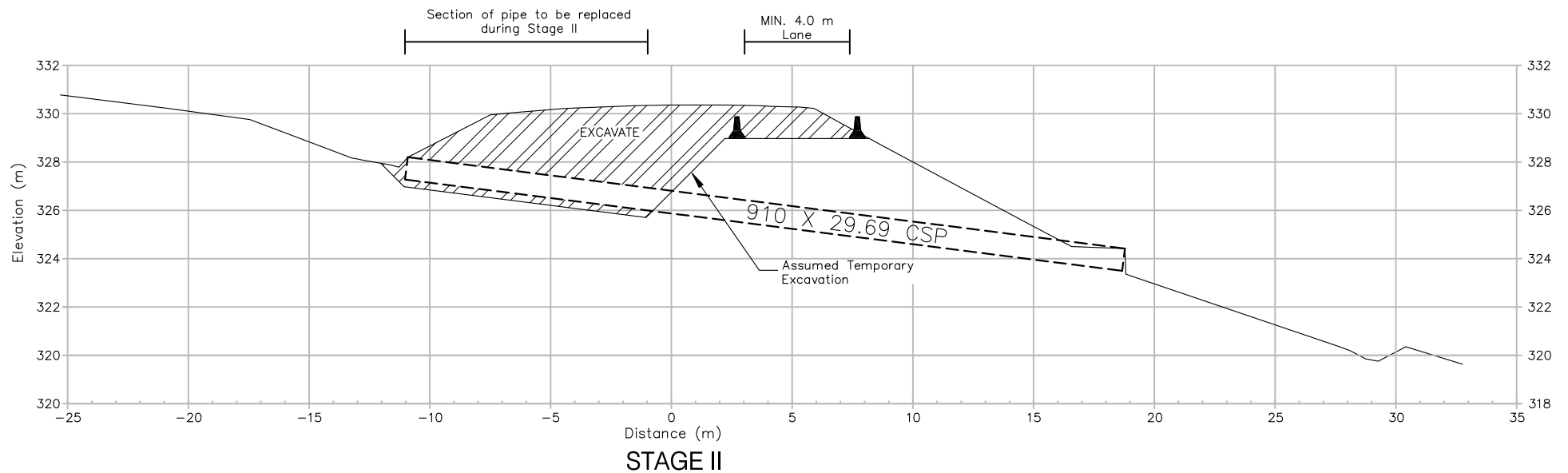
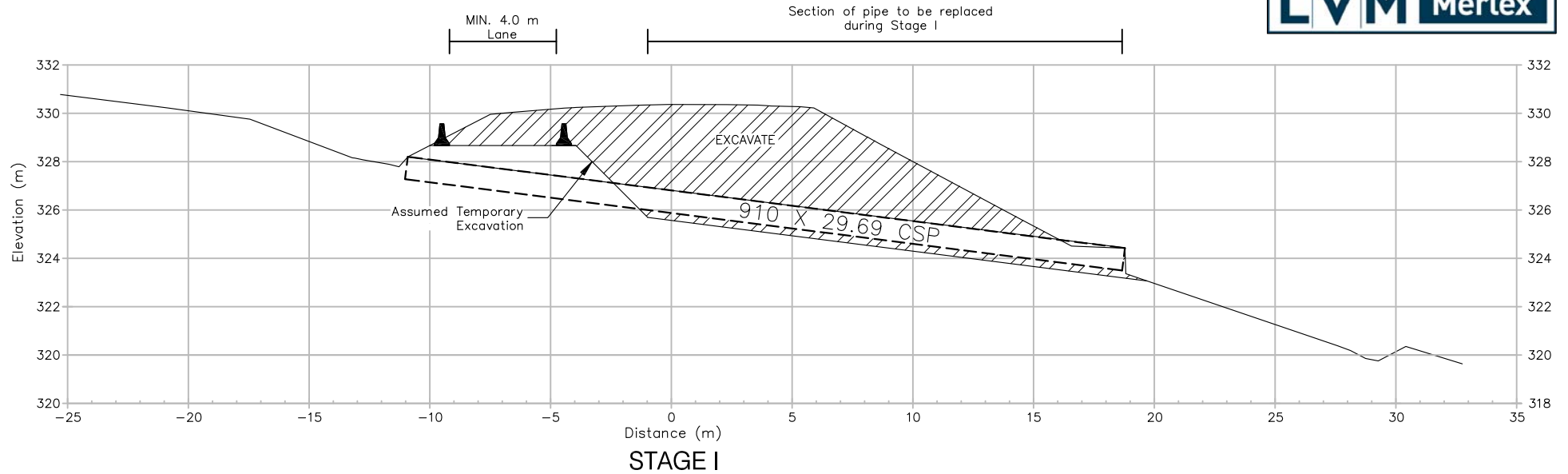
Figure No. S-1:	Slope Stability
Table A:	Comparison of Shoring Alternatives
Figure No. SK-3:	Conceptual Staging Plan
Figure No. SK-4:	Conceptual Shoring Locations
Figure No. SK-5	Conceptual Shoring Sections



Stability Analysis
 Culvert 20+566
 TWP of Chaffey

Table A – Comparison of Shoring Alternatives

Method	Depth Range (m)	Advantages	Disadvantages	Remarks	Estimated Costs
Wood Sheeting	1.5 – 5	-Low cost, -Easily installed in good ground conditions	-Limited by soil conditions, -Limited depth of installation, -Low strength, -discontinuous	Not considered due to rock fill embankment	\$ 650/m ²
Steel Sheet Piles	5 – 21	-High strength, continuous, -Readily available	-Limited by soil conditions (i.e. obstructions)	Not considered due to rock fill embankment	\$ 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)	Not considered due to ground conditions and higher cost	
Soldier piles	5 – 25	-Easy installation -Readily available -Adaptable to various ground conditions	-Pre-drilling may be required -Possible ground loss	Recommended for protection system at this site	\$ 725/m ² Predrilling \$ 1,500/m ²
Tangent/ Secant/ Staggered Drilled Piles	10 – 18	-Readily available -Adaptable to various ground conditions	-Possible ground loss and/or seepage -Poor alignment tolerance	Considered for excavations requiring a protection system at this site	
Concrete Diaphragm	10 – 30	-High Strength -Durable -Can be permanent	-High cost -Requires specialized equipment/control	Not Considered due to ground conditions and higher costs	
Micropiles with reinforced shotcrete face		-Can be installed in various ground conditions -High strength -Good tolerance	-High Cost -Requires specialized equipment	Not recommended to higher cost and method of excavation	\$ 1,200 – 1,500/m ²



METRIC

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

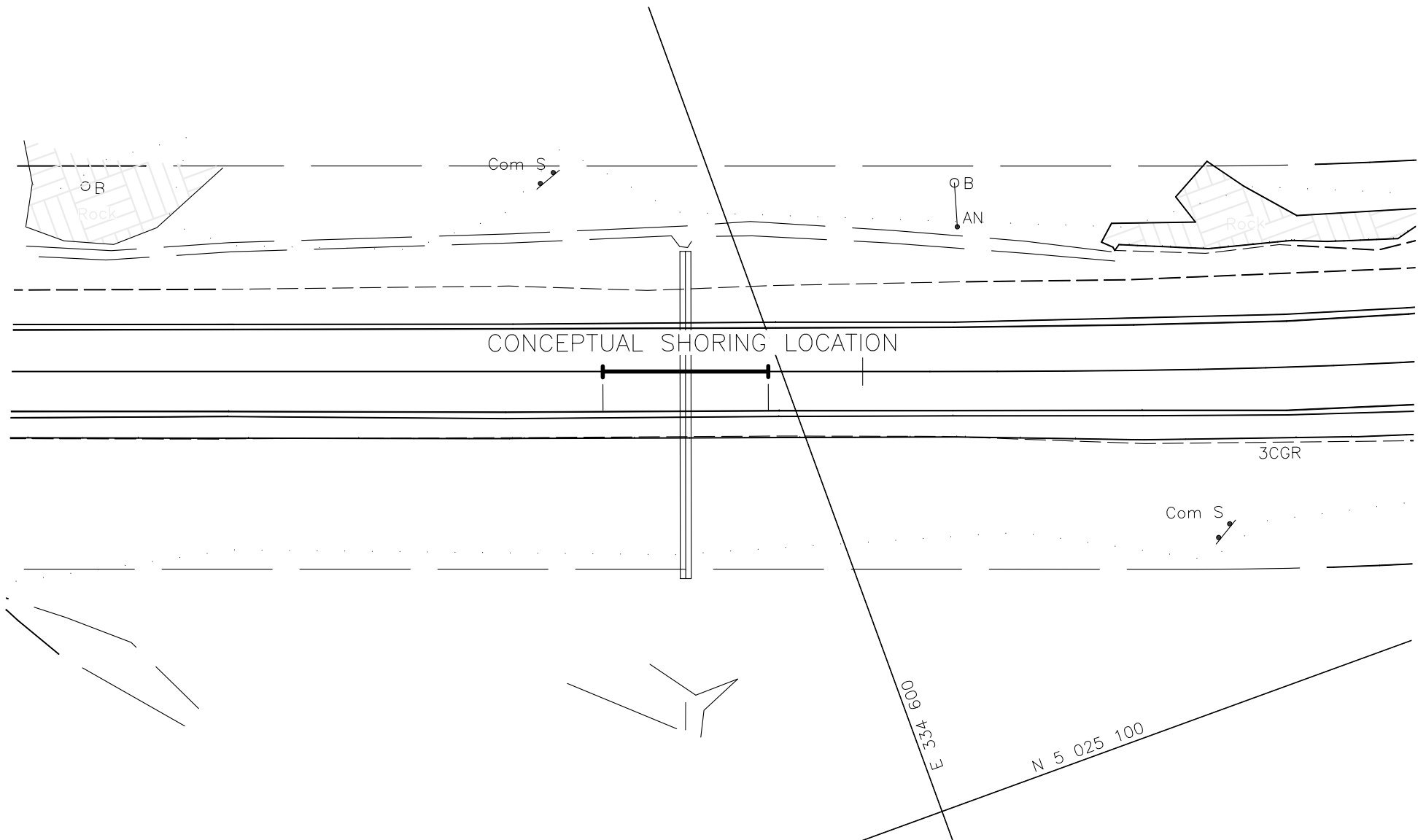
Note:

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



Highway 60, Township of Chaffey - Culvert at Station 20+566
Conceptual Staging Cross-Section

FIGURE SK-3



METRIC

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

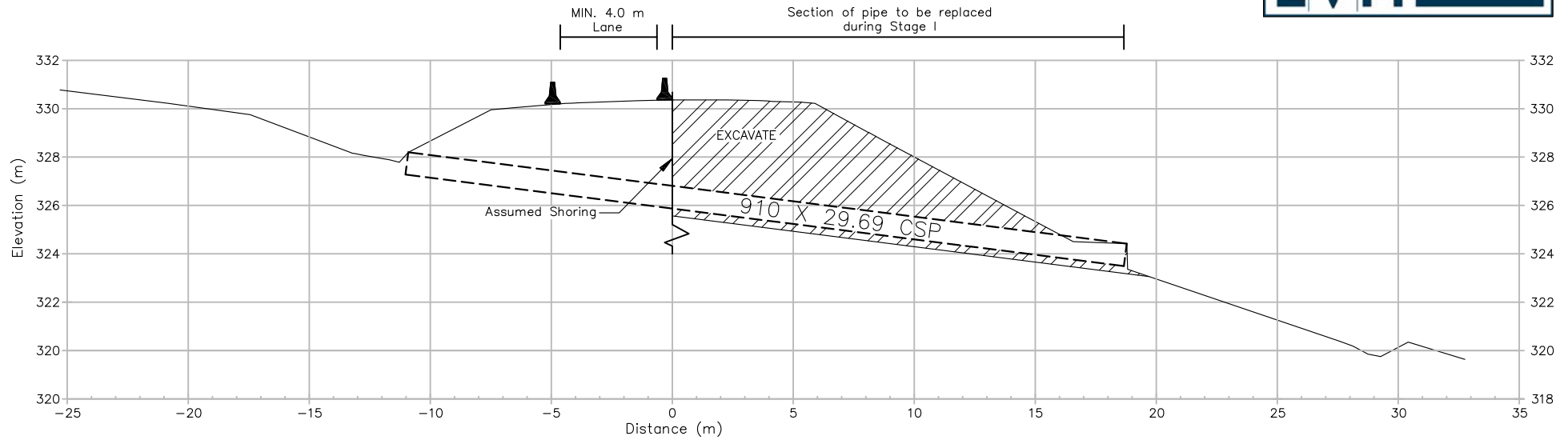
Note:

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

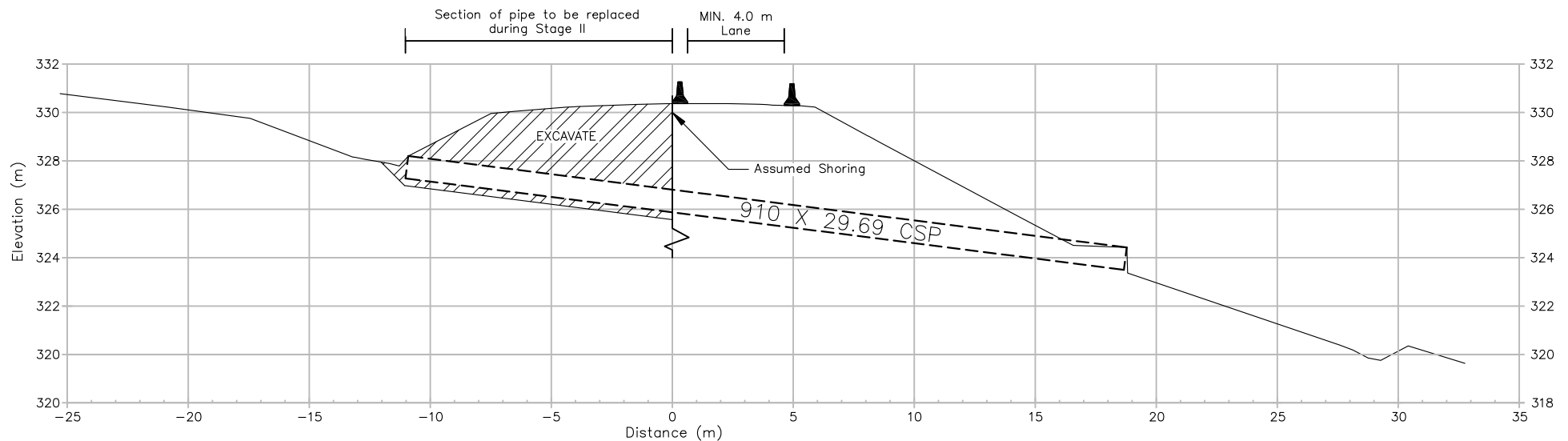


Highway 60, Township of Chaffey - Culvert at Station 20+566
Conceptual Shoring Location Plan

FIGURE SK-4



STAGE I



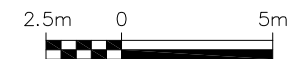
STAGE II

METRIC

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

Note:

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



Highway 60, Township of Chaffey - Culvert at Station 20+566
Conceptual Shoring Cross-Section

FIGURE SK-5