

**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 - Cooper Creek Culvert  
Highway 60  
Station 16+310 – Township of Franklin  
Site No. 42-042/C  
GWP 5333-11-00**

## **FINAL PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT**

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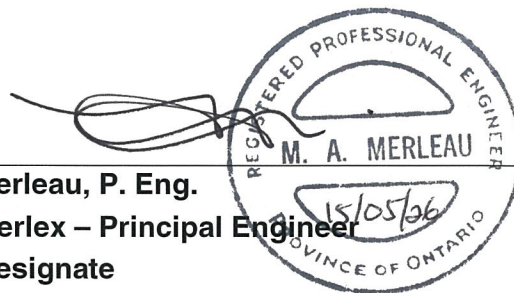


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## 1 INTRODUCTION

LVM-Merlex, a Division of EnGlobe Corp., has been retained by AECOM Canada Ltd., on behalf of the Ministry of Transportation of Ontario (MTO), to carry out a preliminary foundation investigation at the site of an existing centerline culvert (Site No. 42-042/C). The culvert, which allows Cooper Creek to flow through the highway embankment, is located at Station 16+310 in the Township of Franklin on Highway 60, some 1.9 km west of the intersection between Highway 60 and Highway 35 (see Drawing No. 1 in Appendix 1).

The foundation investigation location was specified by the MTO in the Terms of Reference for work under Agreement No. 5013-E-0032: GWP 5333-11-00 for Design-Build. The terms of reference for the scope of work are outlined in LVM-Merlex's Proposal P-14-051 dated May, 2014. The purpose of this investigation was to determine the subsurface conditions in the area of the existing culvert. LVM-Merlex investigated the foundation area by the drilling of boreholes, carrying out in-situ tests, and performing laboratory testing on select samples.

## 2 SITE DESCRIPTION

The single span cast-in-place concrete Rigid Frame Open (RFO) footing culvert is located on Highway 60 at Station 16+310 in the Township of Franklin. At the culvert location, the highway runs locally in a north-south direction. The flow through the culvert is from the east to the west (left to right). A north-south orientation is used in this report for description purposes.

The culvert was constructed in 1963. The concrete RFO culvert at this location has an inside span of 6.22 m in width, with varying wall heights due to the varying exposed bedrock elevations, and is some 28.5 m in total length. The general access into the culvert was difficult because of the varying bedrock surface and the flow of the creek in the summer. Two concrete wingwalls, some 16 metres in total length, exist at skews of some 7° to 12° (relative to the middle of the culvert barrel) at the inlet area. A concrete retaining wall, some 24 m in length, exists parallel to the highway at the outlet end. The culvert is generally in the fair to good condition with hairline to narrow cracking.

The topography at this site is located in a valley area. The existing highway, at the culvert location, is constructed on a granular fill embankment overlying the concrete culvert, with centerline elevation of 327.2 m at the culvert location. The existing embankment extending out from the existing concrete wing/retaining walls, in the area of the culvert, have been built on slope angles of approximately 2.7H:1V to 3H:1V.

### 2.1 SITE PHYSIOGRAPHY AND SURFICIAL GEOLOGY

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

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

### 3 INVESTIGATION PROCEDURES

The fieldwork for this investigation was carried out during the period between August 28<sup>th</sup> and November 4<sup>th</sup>, 2014 during which time three (3) sampled boreholes were advanced. One (1) borehole was advanced through the embankment at the north edge of the existing culvert, and a single borehole was advanced at each of the inlet (east) and the outlet (west) ends of the culverts.

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

Groundwater conditions in the open boreholes were observed during the advancement of and immediately following, completion of the individual boreholes. A single 19 mm diameter standpipe was installed in one open borehole prior to backfilling to allow for further monitoring of the shallow groundwater levels. All open boreholes were backfilled upon completion with compacted auger cuttings in the general order they were removed, and where necessary, bentonite pellet backfill was added to the boreholes to bring them up to grade in accordance with requirements of Ontario Regulation 903. At the borehole through the embankment, the upper portion of the hole, where necessary, was backfilled with an asphalt cold patch to seal the existing asphalt surface.

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

The location of the individual boreholes was determined in the field using highway chainage (established by others) and offset relative to highway centerline. The MTO co-ordinates,

northing and easting, were then established for the boring locations. Elevations contained in this report are referenced to a geodetic datum. The borehole elevations are based on a survey carried out by others.

## **4 SUBSURFACE CONDITIONS**

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

### **4.1 CULVERT SITE NO. 42-042/C, STATION 16+310, TOWNSHIP OF FRANKLIN**

A plan and profile illustrating the borehole locations and stratigraphic sequences is shown on Drawing No. 2, Appendix 3. During the course of the preliminary exploration program, three (3) sampled boreholes were put down at this site, with Borehole No. 1 advanced at the culvert outlet (right side), Borehole No. 2 advanced through the embankment and the north edge of the existing culvert, and Borehole No. 3 advanced at the culvert inlet (left side). Borehole No. 2 was backfilled with the bentonite to seal the borehole properly at and above the top slab of the existing culvert after installing a 19 mm diameter standpipe in the borehole. At the time of the subsurface investigation, the ground surface elevations at Boreholes Nos. 1 to 3 were recorded at elevations 322.8 m, 327.1 m, and 322.1 m, respectively.

#### **4.1.1 Pavement Structure**

Borehole No. 2 was advanced through the embankment where a pavement structure consisting of 102 mm asphalt and 254 mm crushed gravel was penetrated.

#### **4.1.2 Granular Fill**

Underlying the pavement structure at Borehole No. 2 and below ground surface at Borehole No. 3, a layer of granular fill consisting of brown sand gravelly to some gravel, trace silt, and clay was penetrated. The natural moisture contents measured on samples of this deposit recovered in Borehole No. 2 were in the order of 4% to 6%. The natural moisture content was measured at 19% on the one sample recovered from Borehole No. 3. Gradation analyses were carried out on two (2) samples of this deposit, recovered from Borehole No. 2, the results of which indicated 18% to 32% gravel size particles, 60% to 78% sand size particles, and 4% to 8 % silt and clay size particles (Figure No. L-1, Appendix 3). Results of grain size distribution testing carried out on two samples recovered from Borehole No. 2 indicate that the sand fill generally meets requirements of Granular "B" Type I stated in OPS.PROV 1010. Based on SPT 'N'

values of 5 to 23 blows per 300 mm penetration, the compactness of this deposit was described as loose to compact. This deposit was encountered to depths of 3.1 m and 0.7 m below grade at Borehole Nos. 2 and 3 respectively (elevation 324.0 m and 321.4 m, respectively).

#### 4.1.3 Concrete

Underlying the sand fill at Borehole No. 2, the top of the concrete culvert was penetrated. A void was encountered from 3.8 m to 5.2 m below the existing grade, at which point the footing located at the north edge of the existing culvert was penetrated. The underside of the concrete footing was encountered to a depth of 5.9 m below grade at Borehole No. 2 (elevation 321.2 m).

#### 4.1.4 Sand

Underlying the sand fill deposit at Borehole No. 3 and from ground surface at Borehole No. 1, a layer of brown sand, trace to some gravel, some silt and clay was penetrated. The natural moisture content measured on samples of this deposit was in the order of 13% to 22%. Gradation analyses were carried out on two (2) samples of this deposit, the results of which indicated 9% to 18% gravel size particles, 59% to 71% sand size particles, 20% to 23% silt and clay size particles (Figure Nos. L-2 in Appendix 3). Based on SPT 'N' values of 2 to 50 blows per 300 mm penetration, this deposit was described as very loose to dense, generally loose. This deposit was encountered to depths of 0.8 m and 3.2 m below grade at Borehole Nos. 1 and 3, respectively (elevations 322.0 m and 318.9 m, respectively), where bedrock was encountered.

#### 4.1.5 Bedrock

Underlying the above described sands at Borehole Nos. 1 and 3, and at the underside of concrete (footing) encountered at Borehole No. 2, the bedrock was proven by diamond core drilling. The bedrock was described as grey gneiss bedrock. Based on Rock Quality Designation (RQD) values ranging from 47% to 100%, the bedrock was described as poor to excellent quality. Sampling in the bedrock was terminated at depths of 3.8 m, 9.8 m, and 6.7 m below grade at Borehole Nos. 1 to 3, respectively (elevations 319.0 m, 317.3 m, and 315.4 m, respectively). It should be noted that, when encountered, the underlying bedrock surfaces in this area can be very erratic in nature, varying substantially in elevation over short horizontal distances.

### 4.2 GROUNDWATER DATA

At the time of this investigation, the creek water level was measured at elevation 321.0 m at the inlet area on November 3<sup>rd</sup>, 2014. Measurements of the groundwater table and cave-in levels were undertaken, where possible, in the open boreholes during the advance of the individual borings and upon completion. A standpipe was installed in Borehole Nos. 2 and 3 to obtain post borehole completion water levels. These levels are recorded on the individual Record of Borehole Log Sheets (Appendix 2) and shown on the Borehole Locations and Soil Strata Drawing No. 2 in Appendix 3.



The water levels were measured at elevations 319.7 m (August 28<sup>th</sup>, 2014), 321.0 m (September 18<sup>th</sup>, 2014), and 321.8 m (November 4<sup>th</sup>, 2014) at Borehole Nos. 1 to 3, respectively.

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

## 5 DISCUSSION AND RECOMMENDATIONS

### 5.1 GENERAL

A foundation investigation was carried for the proposed replacement of a concrete rigid frame open (RFO) footing culvert, Site No. 42-042/C as identified by the MTO.

The existing culvert, located at Station 16+310 in the Township of Franklin, has an inside span of 6.22 m in width, with heights varying from some 2.6 m to 4.6 m above the founding levels on bedrock, and is some 28.5 m in length. The top of the existing culvert at centerline is at a depth of some 2.8 m (elevation 324.2 m) below centreline. At the inlet area, there are two concrete retaining/wing walls, some 16 metres in total length, established at skews of some 7° to 12° relative to the middle of the culvert barrel. A concrete retaining wall, some 24 m in length, exists parallel to the highway at the outlet area.

The existing highway embankment currently supports two undivided lanes of highway, locally running in a north-south direction at the culvert location. The flow of Cooper Creek 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 at this site has been constructed using a granular pavement structure overlying granular fills. The native material, underlying the embankment fill, generally consisted of very loose to dense, generally loose, sand overlying bedrock.

The RFO structure was widened in 1963 and it is understood that no major rehabilitation has been undertaken since 1963. Based on the requirements stated in RFP, the existing box culvert will be replaced; however it is understood that a technical memorandum dated November 17<sup>th</sup>, 2014 and prepared by AECOM indicates that rehabilitation of two culvert ends is recommended instead of culvert replacement. It is further understood that MTO indicates that the existing culvert is to have a minor patch-repair treatment which is not being considered as “rehabilitation”; this preliminary Foundation Investigation and Design Report (FIDR) is prepared based upon replacing the culvert in accordance with the requirements stated in RFP.

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

### 5.2 FOUNDATION CONSIDERATIONS

The existing culvert is founded on bedrock, based on visual observation and the results of Borehole No. 2. The bedrock below the existing culvert footing and the native sands present below the existing embankment are considered adequate for support of a rigid frame open culvert on bedrock and for a conventional highway embankment of this height on the native sands over bedrock. Bearing resistance, for a rigid frame open type structure, will not be a major issue provided the sound bedrock surface is exposed during construction and the groundwater level is adequately controlled throughout construction, as discussed in Section 5.5.

Flexible culverts (i.e. CSP/HDPE) and/or concrete pipes will not be considered for culvert replacement due to the size of existing culvert at this site. A box type of concrete culvert is not recommended due to the shallow and varying elevations of the bedrock surface. All strip wall footings should be taken down and founded directly on the sound granitic bedrock. Strip footings founded directly on bedrock can be established above the scour depth. The footings can be formed to the shape of the rock and stepped as necessary or the rock levelled (mechanically or by blasting). The bedrock is described as poor to excellent quality, based on RQD data. As such, a geotechnical resistance at ULS not less than 1,000 kPa is appropriate with a minimum footing width of 600 mm. Since the bedrock is essentially an unyielding subgrade, a geotechnical reaction at SLS does not apply.

Considering the somewhat isolated nature of this site, precast concrete footings can also be considered for support of a concrete rigid frame open culvert; however, the bedrock surface would have to be leveled to allow placement of precast footing units. It is recommended that a fill concrete, with a minimum compressive strength of 15 MPa, be used to raise the bedrock surface up to the underside of the precast concrete footings. The fill concrete must be placed within the area of influence of the footing units, which is described as a trapezoid that extends outwards, horizontally from the edges of the foundation, a minimum of 100 mm and then downwards on a 45 degrees outward angle to the bedrock surface.

Adequate shear connection (for example dowels) or key should be considered to connect the mass (fill) concrete to the top of footing. The sliding resistance of mass concrete poured on the top of bedrock may be computed on the basis of an unfactored coefficient of friction of 0.6. This value requires a degree of sliding movement to occur to fully mobilize the resistance.

### 5.2.1 Slope Stability

The maximum height of the embankment above the stream bed at this location is some 5 m on the western side slope and approximately 6 m on the eastern side slope. The inclination angles of the existing slopes are some 3H:1V on the western (right) slope and 1.4H:1V to 2.1H:1V on the eastern (left) slope. Stability analyses, using the GEO-SLOPE computer program, Slope/W (GeoStudio 2007, version 7.17, Geo-Slope International Ltd.), were carried out at this location for the east slope and with existing inclinations and standard embankment slopes of 2H:1V in the granular fill, respectively. For the purposes of these analyses, the materials were modeled using the following parameters:

PARAMETER	MATERIAL	
	GRANULAR FILL	SAND
Unit Weight (kN/m <sup>3</sup> )	19	18.5
Effective Friction Angle (degrees)	30	28

The unit weights and friction angles for the slope calculations are based on general representative values for the various soil types, obtained through laboratory testing and tactile analysis. The results of the analyses indicate factors of safety are in the order of 1.3 for the 2.1H:1V upper side slope out of the existing wing walls, except a marginal condition in the order of 1.0 against the shallow slippage mode at the existing lower west embankment (see Figure No. S-1, Appendix 5). Lower factors of safety will occur during excavation and backfilling as discussed in Section 5.5. Short term stability should not be an issue if construction is carried out as described herein. The long term stability of the new embankment will not be an issue provided it is properly constructed.

### 5.3 CULVERT DESIGN, BEDDING, AND EMBEDMENT

The embankment consists of granular fills. The results of this investigation indicate that, below the base of embankment, the native soils encountered at Boreholes No. 1 and 2, consisted of very loose to dense sands overlying bedrock. A review of the condition of the pavement surface, at the culvert location, revealed minor asphalt cracking; however, in general, the embankment appears to have performed well. The existing embankment has preloaded the soils overlying bedrock at the culvert location and since there will be no change in the height of the embankment, and therefore no increases in embankment load, no appreciable consolidation settlement of the embankment is anticipated. As such, installing the culverts on a camber will not be required at this site.

#### 5.3.1 Rigid Concrete Culvert

The foundation design of the rigid frame open culvert should be in accordance with the MTO Concrete Culvert Design Manual. During backfilling, the granular backfill should be placed in a balanced manner on the outer sides of the RFO culvert. The elevation difference of the backfill on either side of the RFO culvert must be limited to a maximum height of 300 mm. Backfill below the pavement structure and within the depth of frost penetration should be at a minimum granular material of Select Subgrade Material (SSM) or better per OPS.PROV 1010.

Backfilling and construction of pre-cast concrete box culverts shall be in accordance with OPSS 422. During backfilling, the material of bedding, cover and backfill shall be placed in uniform layers not exceeding uncompacted thickness of 200 mm. Backfilling shall be placed in a balanced manner in layers not exceeding 200 mm in thickness on each side of the box unit. The elevation difference of backfilling on either side of the box unit shall be limited to a maximum 400 mm as per OPSS 422. Backfill below the pavement structure and within the depth of frost penetration should be at a minimum granular material of Select Subgrade Material (SSM) or better per OPS.PROV 1010. Cover material for concrete box culverts can consist of Granular A, placed to the dimensions as shown on MTOD-803.021.

The joints between precast box units should be covered with a strip of Non-Woven Class II Geotextile (per OPSS 1860) 600 mm in width, centered over the joint, covering the top of the culvert and extending down the sides of the culvert to prevent the infiltration of fines.

The inlet and outlet stream bed shall be protected with a rip-rap (R-50 size as per OPSS.PROV 1004) apron if the subgrade consists of soil. In this situation the inlet and outlet flow over bedrock and/or constructing wingwalls at the areas of the inlet/outlet, as such the stream bed protection is not required.

## 5.4 CULVERT INSTALLATION AND CONSTRUCTION CONSIDERATIONS

The embankment fill overlying the existing culvert at the centerline of highway is approximately 3 m thick (elevation 324.2 m). The elevation of the top of culvert between the existing inlet and the outlet is essentially flat (some 0.4 m difference). Based on the culvert details shown on historical contract drawing number D-4484-2 (W.P. 209-60) dated November 1961, the founding levels of bedrock varied in elevation from some 319 m to 321 m along the north side of the existing culvert. Accordingly, approximately a 4.5 m to 7 m deep excavation (i.e. to elevations varying from 319 m to 321 m) across the centerline of highway will be required during the construction.

### 5.4.1 Staged Excavation

The present platform width at this location is some 13 m as can be seen on the cross section on Drawing No. 2. The platform width at this location, as is, will not be sufficient to carry out an open excavation in the east-west (left-right) direction using staged construction unless the grade is locally lowered to some 1.6 m below the existing grade of the highway within the excavation areas. Construction using staged sequencing and limiting traffic flow to one lane would be required (see Figure No. SK-2, Appendix 5).

A possible construction plan for a continuous open cut excavation with traffic control operations (temporary traffic lights), as shown on Figure No. SK-2, Appendix 5, is as follows:

- Stage 1
  - Locally lower the grade at the culvert to an elevation of approximately 325.6 m
  - Limit traffic, with a minimum platform width of 6 m one side (left/east) of the centerline of the highway.
  - Remove right (west) embankment to the north and south, slope as required, and remove right (west) section of the existing culvert.
  - Construct the new culvert to the right (west), about 14 m in length, with adequate backfilling as stated in Section 5.3.1.
  - Reconstruct the embankment on the right (west), up to a minimum platform width of 6 m for traffic.
- Stage 2
  - Divert the single lane of traffic to the right (west) and continue open excavation to install the remainder of the culvert on the left (east).

- As the width of the platform increases on the left, the vertical alignment can be raised to final grades, and the traffic can revert back to two lanes when sufficient width permits.

#### 5.4.2 Protection System

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

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

The installation of a protection system for use in the culvert replacement operation will require penetration through some 3 m of granular fills, at the single borehole location, above the top of the existing culvert. As noted, bedrock was encountered at relatively shallow depths at this culvert location, which may result in difficulty adequately fixing the toe of the steel sheet piles to resist the active pressure of the backfill. As such, if a temporary flexible wall is required for protection it would be more appropriate to use a wall which requires drilled support members to penetrate bedrock at the base of excavation. This could consist of a staggered drilled pile wall or micropiles with reinforced shotcrete face, however these shoring systems are generally more costly.

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 and a schematic cross section are illustrated on Figure Nos. SK-3 and SK-4, Appendix 5.

Considering the relative shallow depth of bedrock, and provided a shoring wall that is sufficiently embedded/fixed below excavation base is used, a waler and raker may be used to span the width of the culvert; however, a tieback system may also be chosen by the contractor. If tiebacks are used, 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):

$$R = \sigma_z' A_s L_s \alpha_g$$

Where:  $\sigma_z'$  = effective vertical stress at the midpoint of the load carrying fixed length

$A_s$  = effective unit surface area of the anchor

$L_s$  = effective embedment length of the anchor

$\alpha_g$  = 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.

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

$K_a$  = active earth pressure coefficient, as described in Section 5.6,

$\gamma$  = 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.

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

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

## 5.5 EXCAVATION, DEWATERING, AND EMBANKMENT CONSTRUCTION

All temporary excavations greater than 1.2 m in depth must, at a minimum, be sloped or shored in accordance with the Occupational Health and Safety Act Regulations for Construction Projects. The embankment material, above the water table, is considered a Type 3 soil as defined in the Occupational Health and Safety Act and Regulations for Construction Projects. Temporary open excavations above the groundwater table, could be cut back at an angle of 1H:1V, provided they are monitored continuously; however, below the groundwater table, the side slopes in fill and/or native materials will have to be cut back to an angle of 2H:1V, possibly flatter, dependent upon the Contractors' chosen method of controlling the groundwater.

Excavations must be maintained in a dewatered condition during excavation and foundation construction, and every effort must be made to prevent disturbing (piping/boiling) at the founding subgrade. Groundwater control, in accordance with OPSS 517 and 518, will be required to maintain a stable subgrade during culvert installation.

The water levels in the creek were measured at elevation some 321.7 m at the inlet area and 321.9 m at the outlet area, respectively, at the time of this investigation. The groundwater

levels were measured at elevations 321.7 m, 321.8 m, and 321.8 m at Borehole Nos. 1 to 3, respectively, at the time of this investigation. The bedrock profile, based on Drawing No. D-4484-2 (W.P. 209-60) dated November 1961, indicates the bedrock was encountered at approximately Elevation 321 m, dropping to about Elevation 319 m over the last 4 m of culvert at the west (outlet) end. Considering the minimal head of water that may have to be controlled during construction it is likely that the contractor will employ the conventional dewatering method of pumping from filtered sump holes located in the base of the excavation.

In order to dewater the culvert location, a cofferdam will be required at both the inlet and outlet. A temporary gravity type cofferdam is the recommended method of controlling the creek flow. A gravity type cofferdam can be constructed of earth fill with a low permeable core, sand bag/metre bag, or aquadam (water filled bladder) type dam. Depending upon the base width of the cofferdam seepage may develop. This may require groundwater control by pumping from filtered sump holes within the excavation area. Sufficient temporary bypass pipes need to be installed through the existing embankment, or possibly through the existing culvert, for flow diversion. Ultimately, the method of excavation, dewatering, and stream flow diversion will be the choice of the contractor; however the importance of maintaining the subgrade in a dewatered stable condition during excavation and construction operations cannot be stressed enough.

#### 5.5.1 **Bedrock Excavation**

Bedrock was encountered at a relatively shallow depth below the top of existing culvert and the ground surface at the locations of the three boreholes. Depending on the location and founding levels of the new culvert, bedrock may be within the anticipated depth of excavation. Therefore bedrock excavation/hoe ramming and/or blasting operations may be required. The existing footings of the culvert may be demolished and replaced by the new concrete footing per design requirements.

If blasting is required reference shall be made to OPSS 120. A blast design is required to be provided by the blasting contractor before blasting operations are carried out. A pre-blast survey (OPSS 120.07.03) and/or blast monitoring may be required considering the location of the commercial operation relative to the culvert location.

### 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
Unit Weight (kN/m <sup>3</sup> )	22.8	21.2	19	18.5
Angle of Internal Friction	34°	31°	30°	28°
Coefficient of Active Earth Pressure ( $K_a$ )	0.28	0.32	0.33	0.36
Coefficient of Passive Earth Pressure ( $K_p$ )	3.54	3.12	3.0	2.77
Coefficient of Earth Pressure at Rest ( $K_o$ )	0.44	0.48	0.5	0.53

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

## 5.7 CONSTRUCTION CONCERNS

Considering the nature of the granular fill embankment, no unusual construction concerns are anticipated if construction is carried out in general conformance with the above discussion. It is noted that bedrock was encountered at relatively shallow depth during this investigation. Therefore, design and construction considerations will need to account for the shallow bedrock which will affect the design of temporary protection systems, dewatering system, and the methodology of construction (i.e. bedrock removal for levelling, use of concrete for levelling, shaping foundations to the bedrock surface, etc.), and creek diversion.

## 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, at this preliminary design stage. 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 4 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

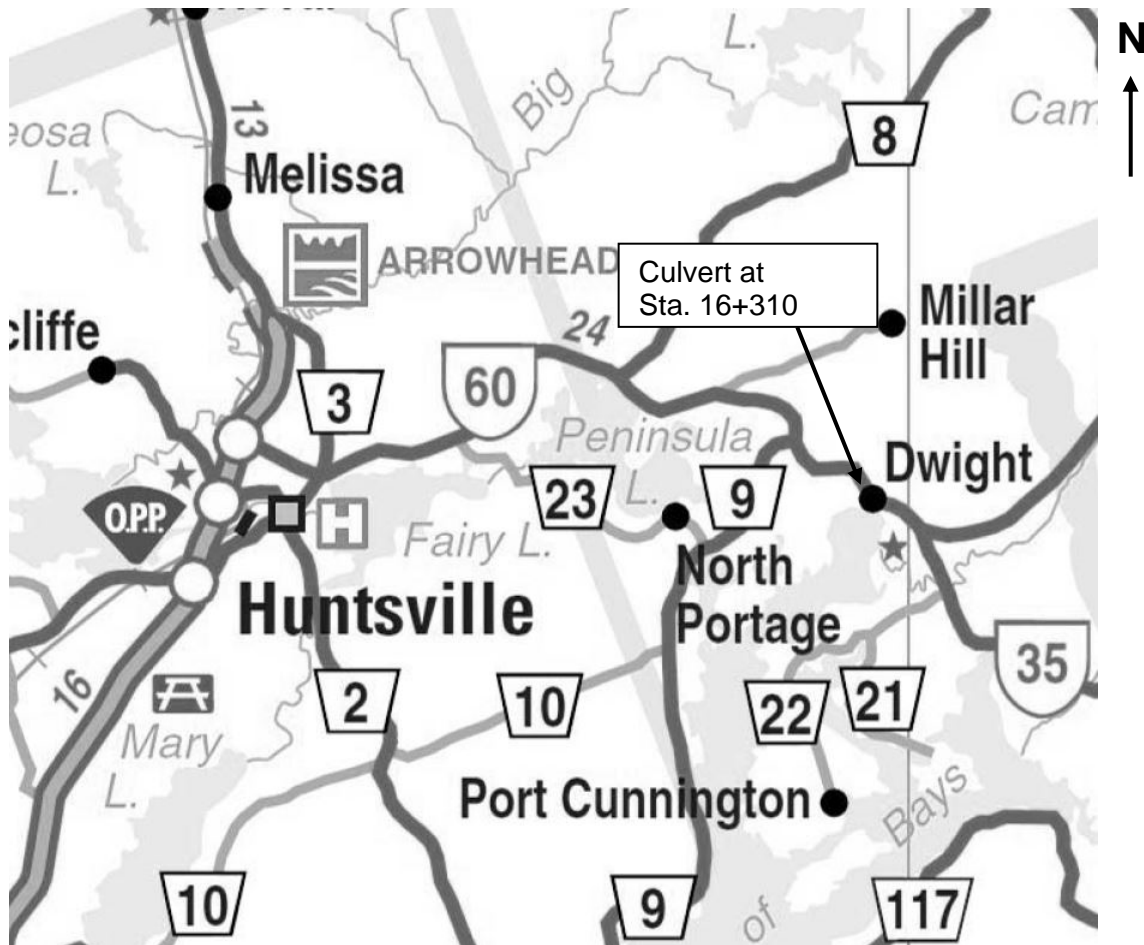
Drawing No. 1

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 16+310 Culvert

Site No. 42-042/C

Township of Franklin



Reference No: 14/07/14083-F7

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
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 5021570.3 E 342800.2 - Franklin Twp., Station 16+322 ORIGINATED BY JL

PROJECT GWP 5333-11-00, Highway 60 - F7 BOREHOLE TYPE Track Mounted CME 45 - Hollow Stem Augers COMPILED BY SH

CLIENT AECOM DATE (Started) 28 August 2014 TIME (Completed) 12:30:00 PM CHECKED BY MAM

DATE (Completed) 28 August 2014

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT			LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100	W <sub>p</sub>	W	W <sub>L</sub>			
322.8	Ground Surface		1	SS	5													
0.0	SAND, trace gravel, some silt and clay brown (compact)																	
322.0	Auger Refusal Start rock coring		2	RC	Rec=100% RQD=88%													
0.8	BEDROCK - grey gneiss good to excellent quality																	
			3	RC	Rec=100% RQD=98%													
319.0	End of Sampling End of Borehole																	
3.8																		

COMMENTS		WATER LEVEL RECORDS	
+ 3, × 3 : Numbers on right refer to Sensitivity Numbers on left refer to values greater than 120 kPa ○ 3% STRAIN AT FAILURE	Date (dd/mm/yy)/Time	Water Depth (m)	Cave In (m)
	1) 28/8/14 12:30:00 PM	3.07	3.68
	2)	-	-
	3)	-	-

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

MEL-GEO 14083 - BOREHOLE LOGS - F7.GPJ MEL-GEO.GDT 6/3/15



**METRIC****RECORD OF BOREHOLE NO. 2**

REFERENCE 14/07/14083 DATUM Geodetic LOCATION N 5021588.2 E 342808.4 - Franklin Twp., Station 16+308 ORIGINATED BY JL

PROJECT GWP 5333-11-00, Highway 60 - F7 BOREHOLE TYPE Truck Mounted CME 45 - Hollow Stem Augers COMPILED BY SH

CLIENT AECOM DATE (Started) 16 September 2014 TIME   
DATE (Completed) 18 September 2014 (Completed) 10:35:00 AM 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 $\gamma$	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100	W <sub>p</sub>	W	W <sub>L</sub>		
327.1	Ground Surface																
0.0	102 mm Asphalt 254 mm Crushed Gravel  FILL - sand, gravelly to some gravel, trace silt and clay  brown  (compact/loose)		1	SS	23											32 60 (8)	
			2	SS	21												
			3	SS	10											18 78 (4)	
			4	SS	5												
324.0																	
3.1	Auger Refusal start rock coring		5	RC	Rec=50%												
323.3	Concrete (top of culvert)																
3.8	Void																
321.9																	
5.2	Concrete (culvert footing)																
321.2																	
5.9	BEDROCK - grey gneiss  poor to good quality		6	RC	Rec(BDR)=77% RQD(BDR)=47%												
			7	RC	Rec=92% RQD=79%												
			8	RC	Rec=100% RQD=86%												
317.3																	
9.8	End of Sampling End of Borehole																

**COMMENTS**

Void was dry and flowing water sound was not heard. Paper bag installed to plug annular space between 25 mm diameter PVC pipe and NQ hole in upper concrete. Backfill using bentonite above paper bag to top of PVC pipe located at 50 mm below grade.

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

**WATER LEVEL RECORDS**

Date (dd/mm/yy)/Time	Water Depth (m)	Cave In (m)
1) 18/9/14 10:40:00 AM	6.15	▽ -
2)	-	▽ -
3)	-	▼ -

MEL-GEO 14083 - BOREHOL LOGS - F7.GPJ MEL-GEO.GDT 6/3/15

**METRIC****RECORD OF BOREHOLE NO. 3**

REFERENCE 14/07/14083 DATUM Geodetic LOCATION N 5021604.9 E 342827.8 - Franklin Twp., Station 16+299 ORIGINATED BY JL

PROJECT GWP 5333-11-00, Highway 60 - F7 BOREHOLE TYPE Track Mounted CME 45 - Hollow Stem Augers COMPILED BY SH

CLIENT AECOM DATE (Started) 3 November 2014 TIME

DATE (Completed) 4 November 2014 (Completed) 9:10:00 AM 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 $\gamma$	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100					
322.1	Ground Surface																
0.0	100 mm forest mat FILL - sand, some gravel, trace silt		1	SS	5												
321.4	SAND, trace to some gravel, with silt and clay occasional grass rootlets dark brown		2	SS	2												
0.7	brown (loose/dense)		3	SS	5												
			4	SS	50												
318.9	Auger Refusal Start rock coring		5	RC	Rec = 97% RQD = 78 %												
3.2	BEDROCK - grey gneiss good to excellent quality		6	RC	Rec = 98% RQD = 100 %												
315.4			7	RC	Rec = 100% RQD = 100 %												
6.7	End of Sampling End of Borehole																

COMMENTS		WATER LEVEL RECORDS	
+ 3, × 3 : Numbers on right refer to Sensitivity Numbers on left refer to values greater than 120 kPa ○ 3% STRAIN AT FAILURE		Date (dd/mm/yy)/Time	Water Depth (m)
		1) 4/11/14 9:15:00 AM	0.41
		2) 4/11/14 10:15:00 AM	0.33
		3)	-

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

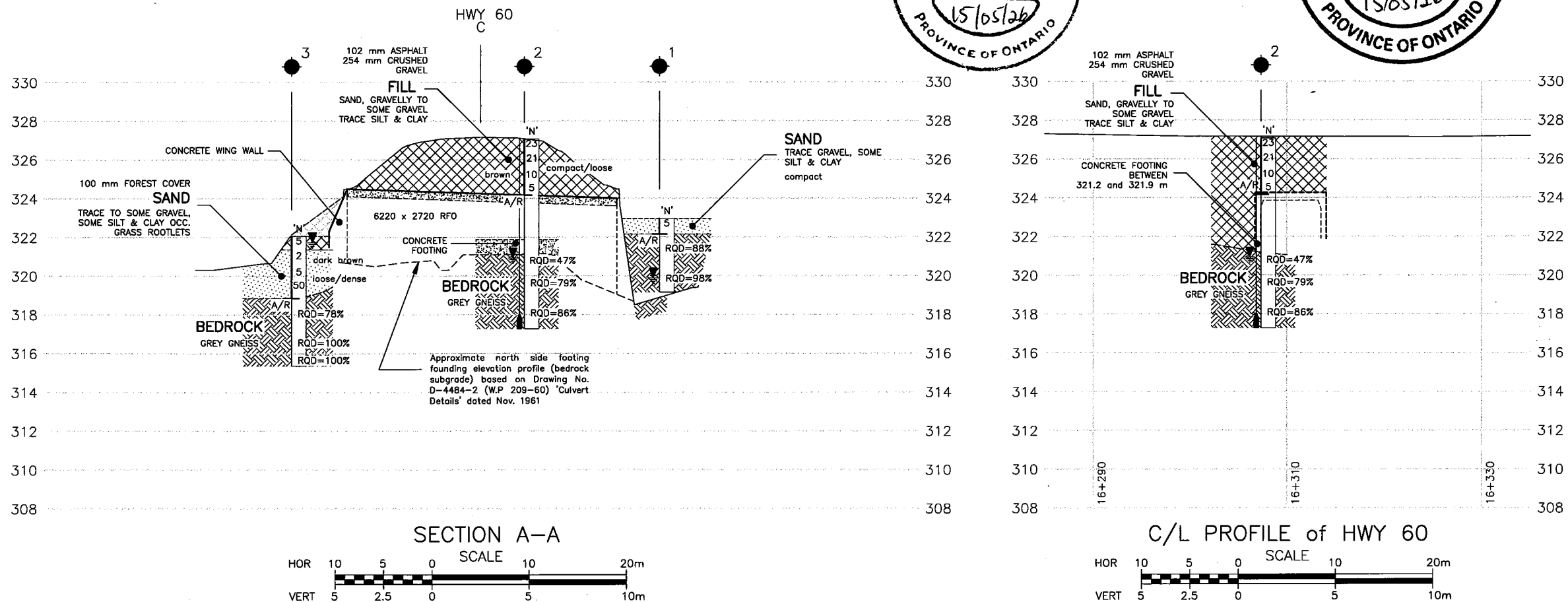
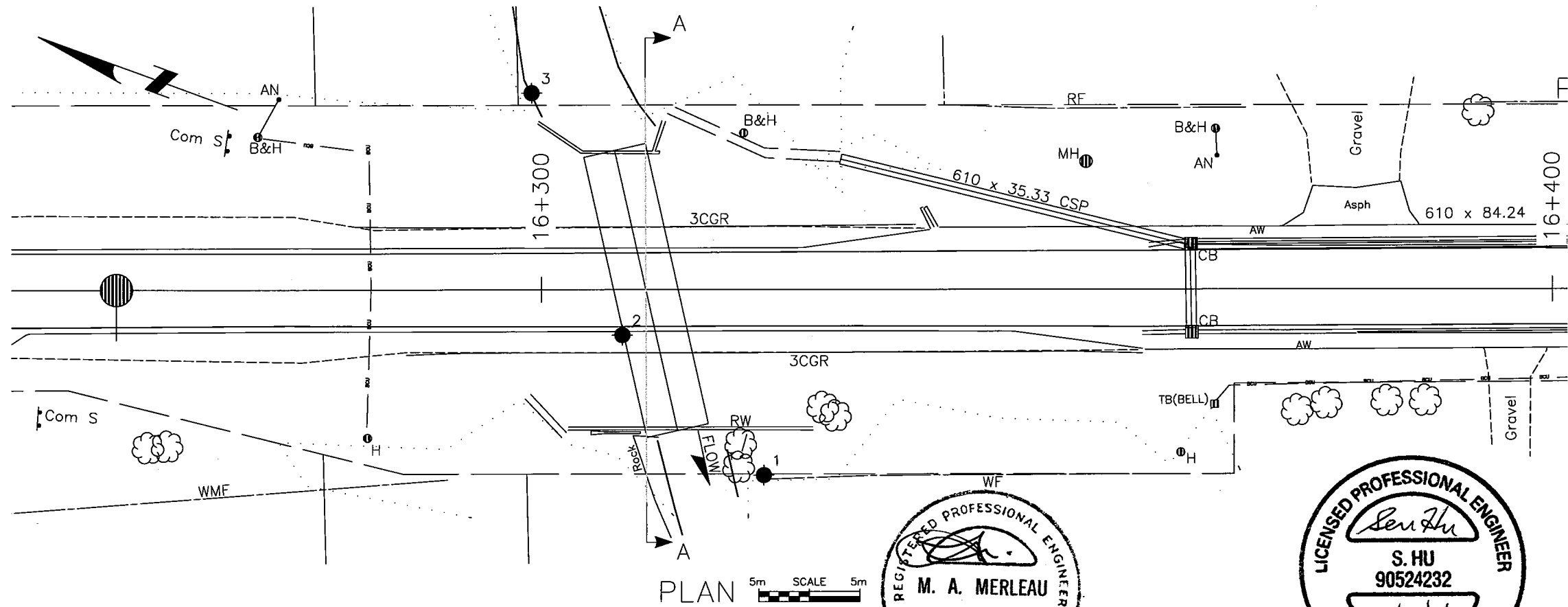
MEL-GEO 14083 - BOREHOLE LOGS - F7.GPJ MEL-GEO.GDT 6/3/15

## **Appendix 3     Borehole Plan and Lab Data**

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

CAD FILE LOCATION AND NAME: E:\04083 - PAV & FDN, Hwy 60, Huntsville & Hwy 113, Cobalt (GEOCHN) FOUNDATIONS Drawings\F7 Working - Do Not Move or Delete Files\14083-F7 - Drawing Package, 16+310 Franklinburg  
MODIFIED: 2/26/2015 11:47:13 AM BY: GRASSER  
DATE PLOTTED: 2/26/2015 11:58:31 AM BY: RYAN GRASSER

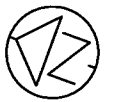
MINISTRY OF TRANSPORTATION, ONTARIO  
PR-D-707 04-05



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 16+310  
FRANKLIN TOWNSHIP

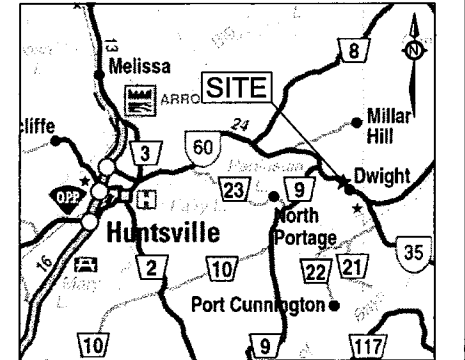
DRAWING

BOREHOLE LOCATIONS  
AND SOIL STRATA

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	322.8	25.5m Rt	5021570.3	342800.2
2	327.1	12.5m Lt	5021588.2	342808.4
3	322.1	4.4m Lt	5021604.9	342827.8

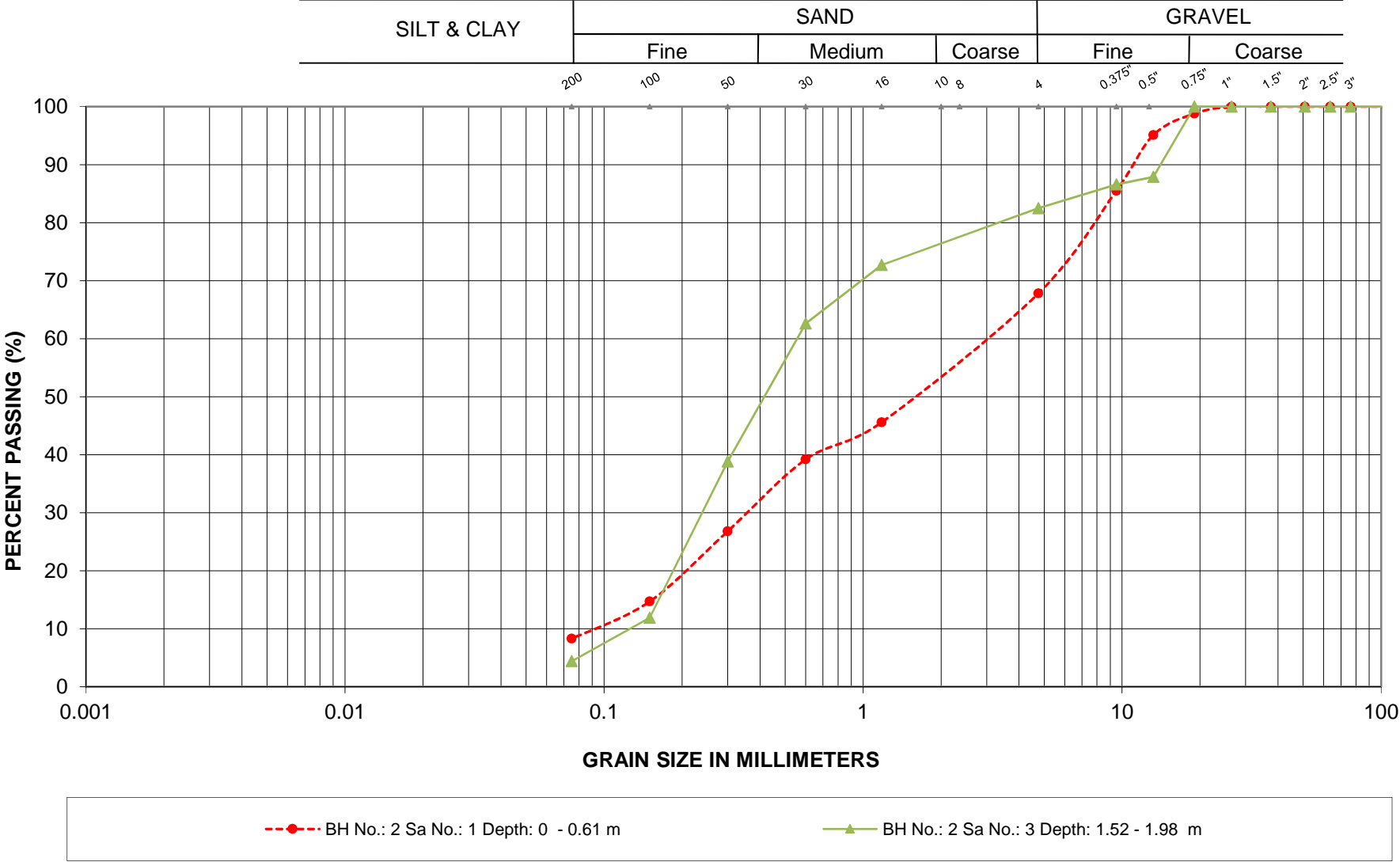
NOTES:  
1. The boundaries between soil strata have been established at the borehole locations only. The boundaries illustrated and stratigraphy between boreholes on this drawing are assumed based on borehole data and may vary. They are intended for design only.  
2. Base plan and alignment provided in digital format by exp. on October 28, 2014.

GEOCRES No. 31E-349

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



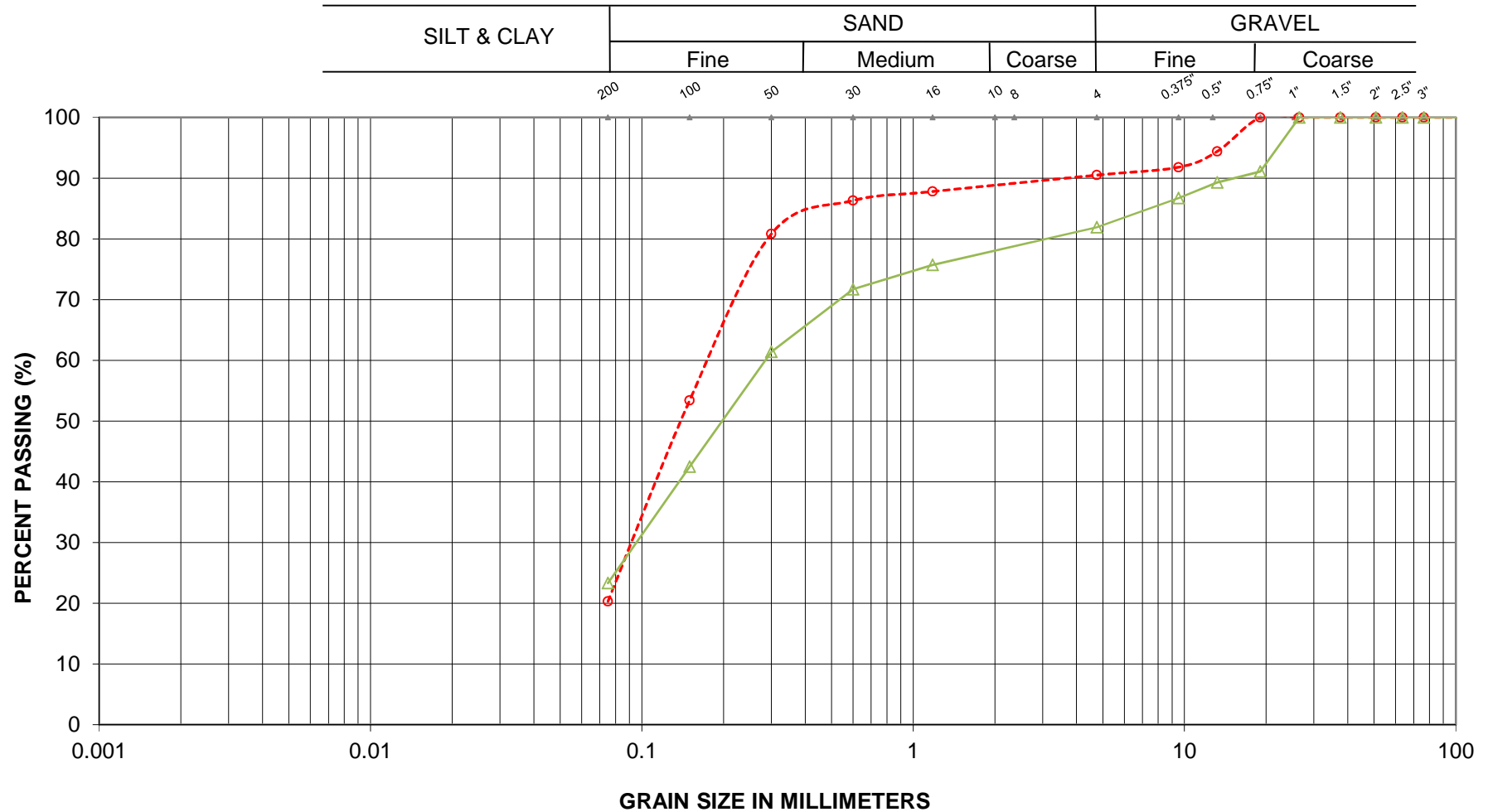
SAND FILL

LOCATION: Hwy 60 Sta. 16+310 Culvert  
TWP. Franklin, Ontario

LVM-Merlex, a Division EnGlobe Corp.

FIGURE L-1

# GRAIN SIZE ANALYSIS



---○--- BH No.: 1 Sa No.: 1 Depth: 0 - 0.61 m

—△— BH No.: 3 Sa No.: 4 Depth: 2.29 - 2.74 m

SAND

LOCATION: Hwy 60 Sta. 16+310 Culvert  
TWP. Franklin, Ontario

LVM-Merlex, a Division EnGlobe Corp.

FIGURE L-2

## Laboratory Tests - Summary Sheet



Borehole No.	Sample No.	Depth	Grain Size Analysis				NMC	Atterberg Limits			SPT 'N'	USCS	Unit Weight (kN/m3)	Remarks
			Gravel Size (%)	Sand Size (%)	Silt Size (%)	Clay Size (%)		LL (%)	PL (%)	IP (%)				
1	1	0.3	9	71	20		13.2				5			
2	1	0.2	32	60	8		3.5				23			
	2	1.0					5.9				21			
	3	1.8	18	78	4		4.2				10			
	4	2.5					4.2				5			
3	1	0.3					18.9				5			
	2	1.0					22.4				2			
	3	1.8					20.6				5			
	4	2.5	18	59	23		15.7				50			

## Appendix 4    Photo Essay

Enclosure No. 5:

Photo Essay



Culvert Inlet – Looking Southeast

Photo: 1



Upstream at Culvert Inlet – Looking East

Photo: 2



Project: Hwy 60 – Cooper Creek Culvert 16+300

Photos Provided By: LVM

Date: November 2014

Culvert Outlet – Looking Northwest

Photo: 3



Downstream at Culvert Outlet – Looking West

Photo: 4



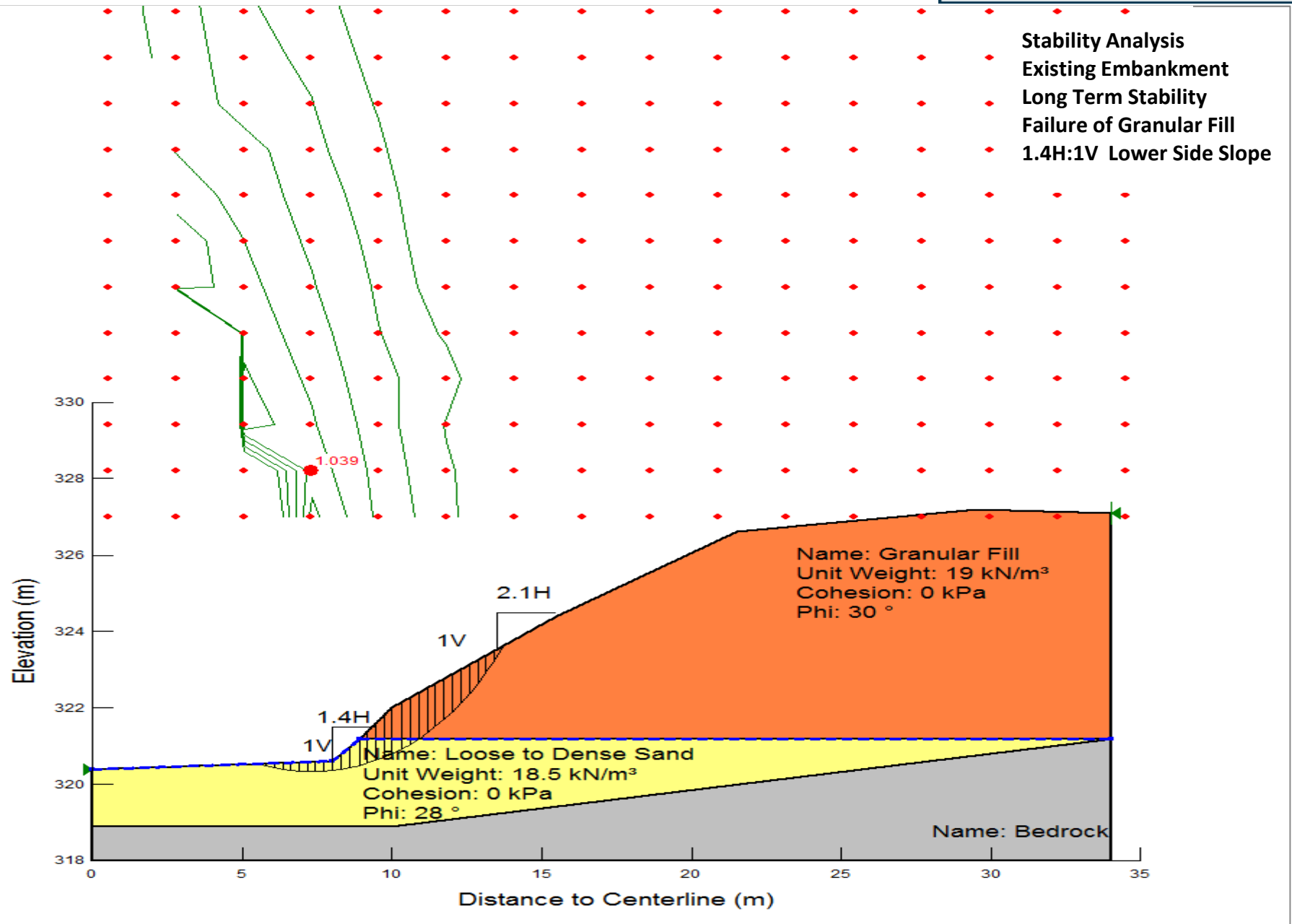
Project: Hwy 60 – Cooper Creek Culvert 16+300

Photos Provided By: LVM

Date: August 2014

## Appendix 5    Design Data

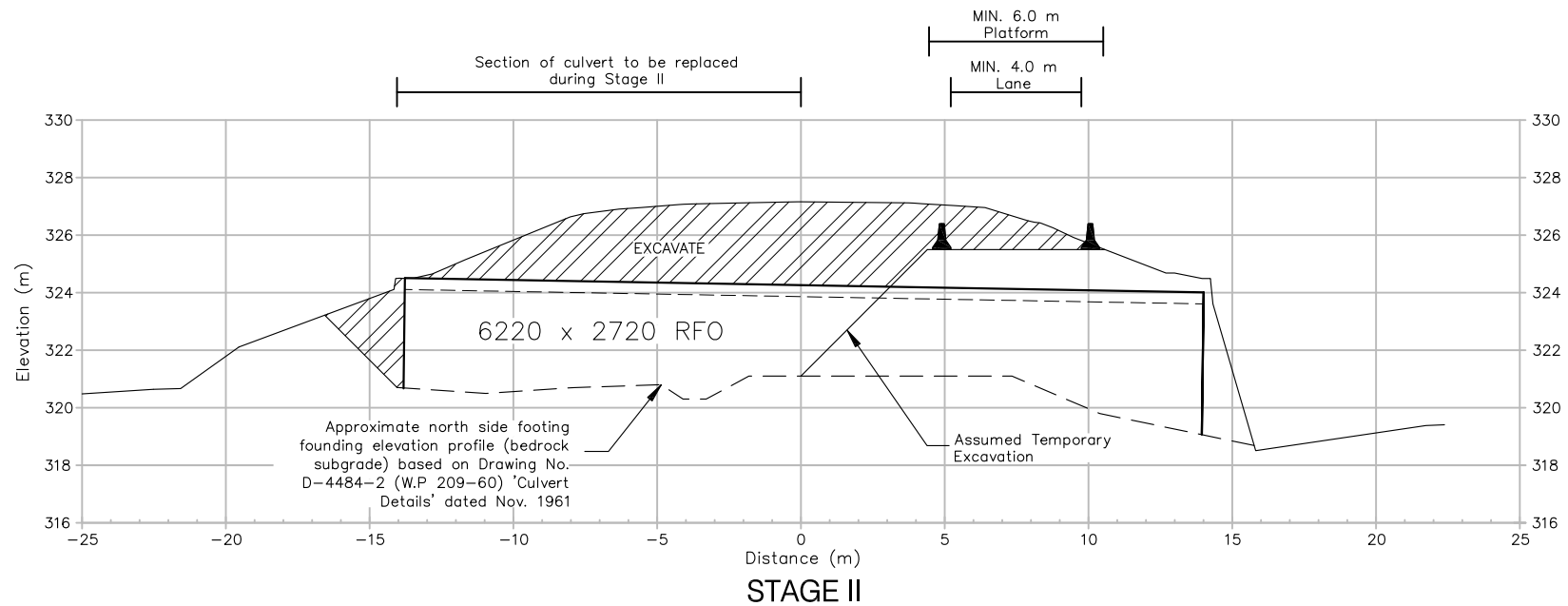
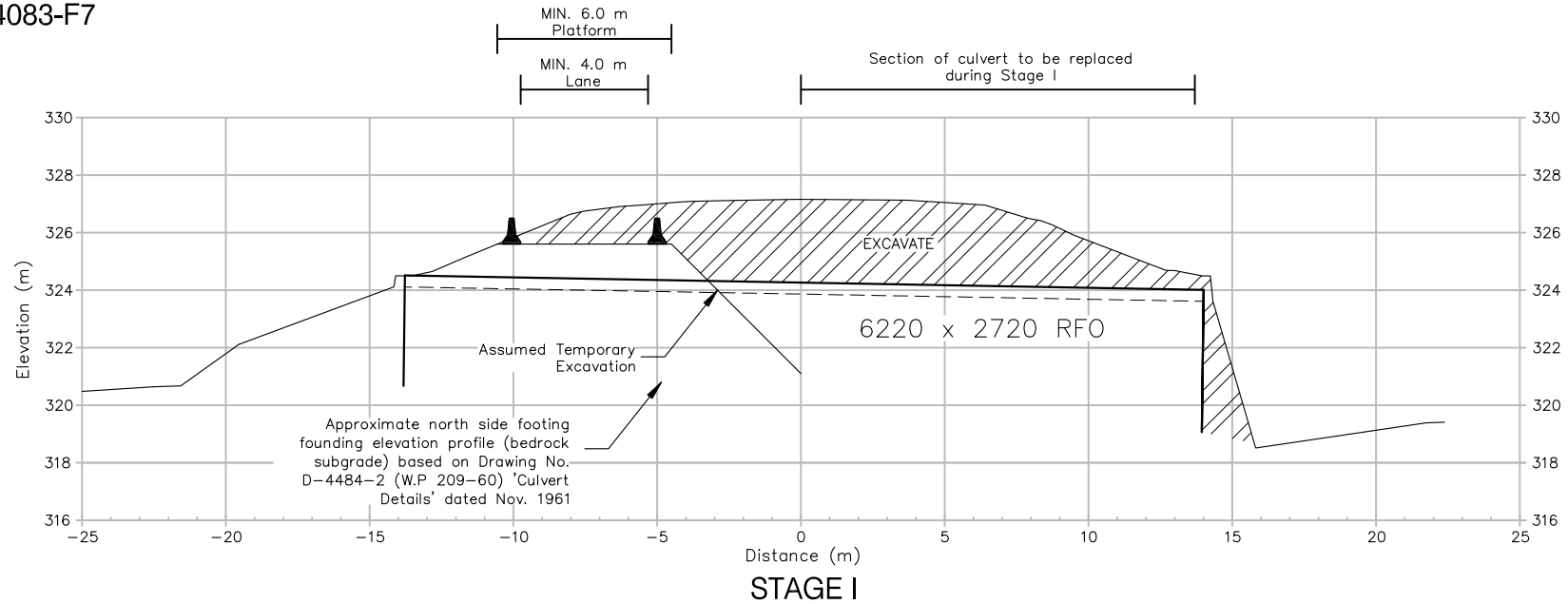
Figure No. S-1:	Slope Stability Analyses
Table A:	Comparison of Shoring Alternatives
Figure No. SK-2:	Conceptual Staging Excavation Cross Section
Figure No. SK-3:	Conceptual Shoring Location Plan
Figure No. SK-4:	Conceptual Shoring Cross Section





**Table A – Comparison of Shoring Alternatives**

Method	Depth Range (m)	Advantages	Disadvantages	Remarks	Estimated Costs
Wood Sheeting	1.5 – 5	-Low cost, -Easily installed in good ground conditions	-Limited by soil conditions, -Limited depth of installation, -Low strength, -discontinuous	Not recommended due to high groundwater and permeable granular fills	\$ 650/m <sup>2</sup>
Steel Sheet Piles	5 – 21	-High strength, continuous, -Readily available	-Limited by soil conditions (i.e. obstructions)	Not recommended, difficulty in breaking the concrete box and penetrate sufficient embedment depth into the bedrock	\$ 650/m <sup>2</sup>
Pre-cast concrete panels	3 – 10	-Durable -Assists in minimizing seepage	-Limited depths -Can be damaged during installation -Limited by soil conditions (i.e. obstructions)	Feasible, but higher cost	
Soldier piles	5 – 25	-Easy installation -Readily available -Adaptable to various ground conditions	-Pre-drilling may be required -Possible ground loss	Feasible with predrilling sufficient embedment depth into the bedrock and adequate dewatering	
Tangent/ Secant/ Staggered Drilled Piles	10 – 18	-Readily available -Adaptable to various ground conditions	-Possible ground loss and/or seepage -Poor alignment tolerance	Feasible with sufficient embedment depth into 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 with sufficient embedment depth into the bedrock and adequate dewatering	\$ 900/m <sup>2</sup>

**METRIC**

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

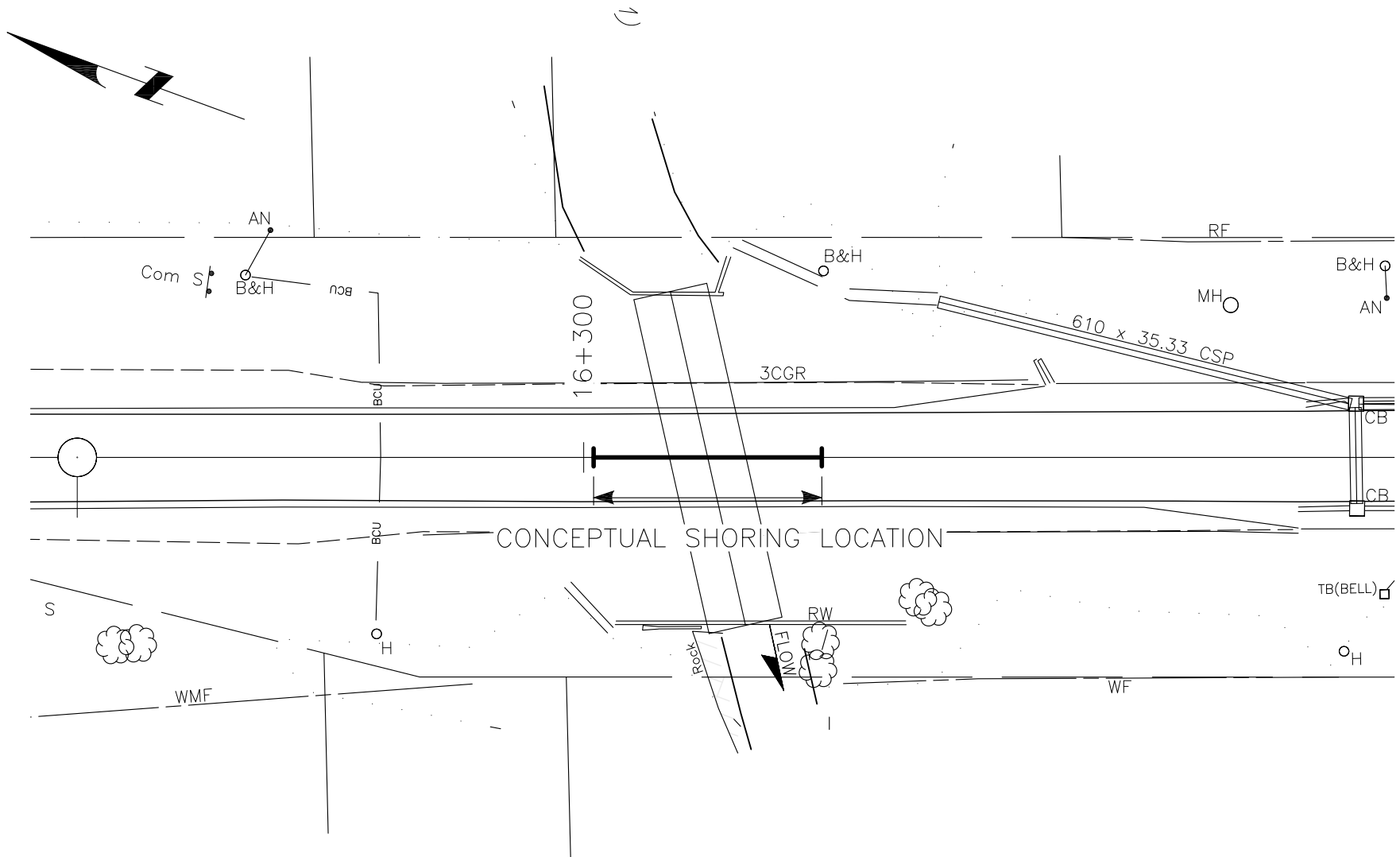
Note:

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



Highway 60, Township of Franklin - Culvert at Station 16+310  
Conceptual Staging Cross-Section

FIGURE SK-2



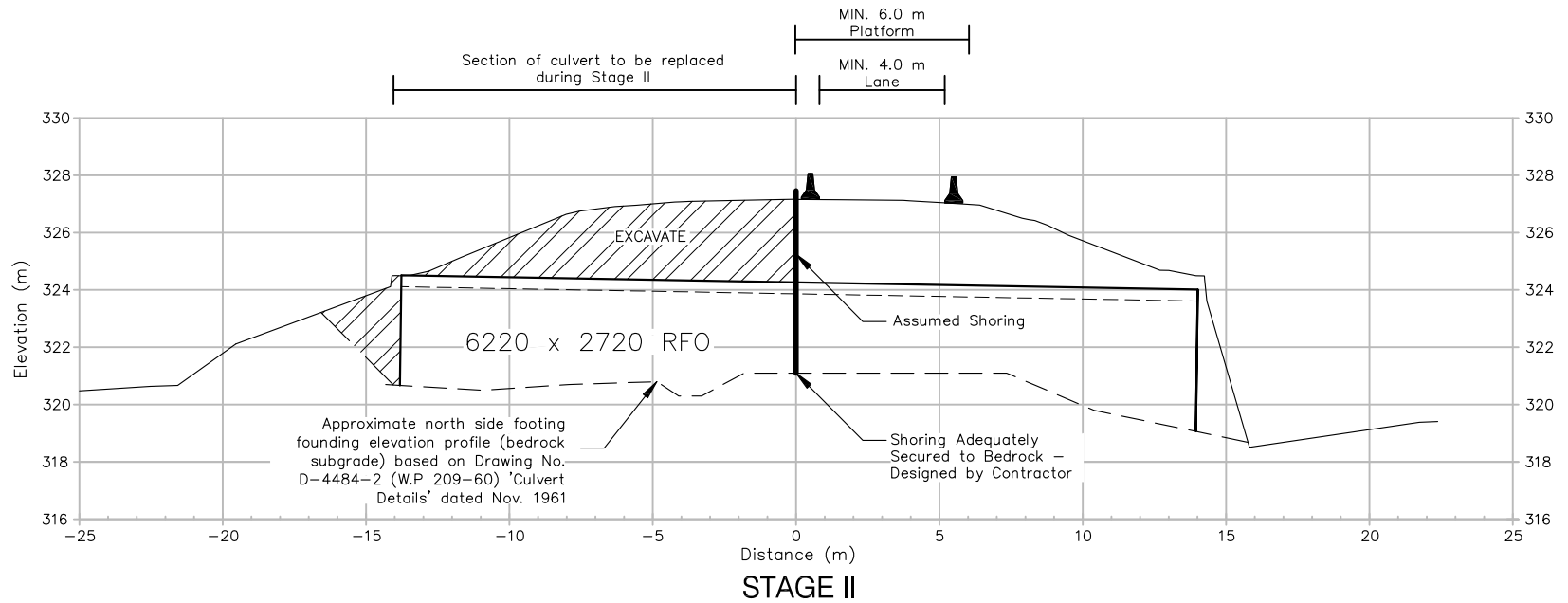
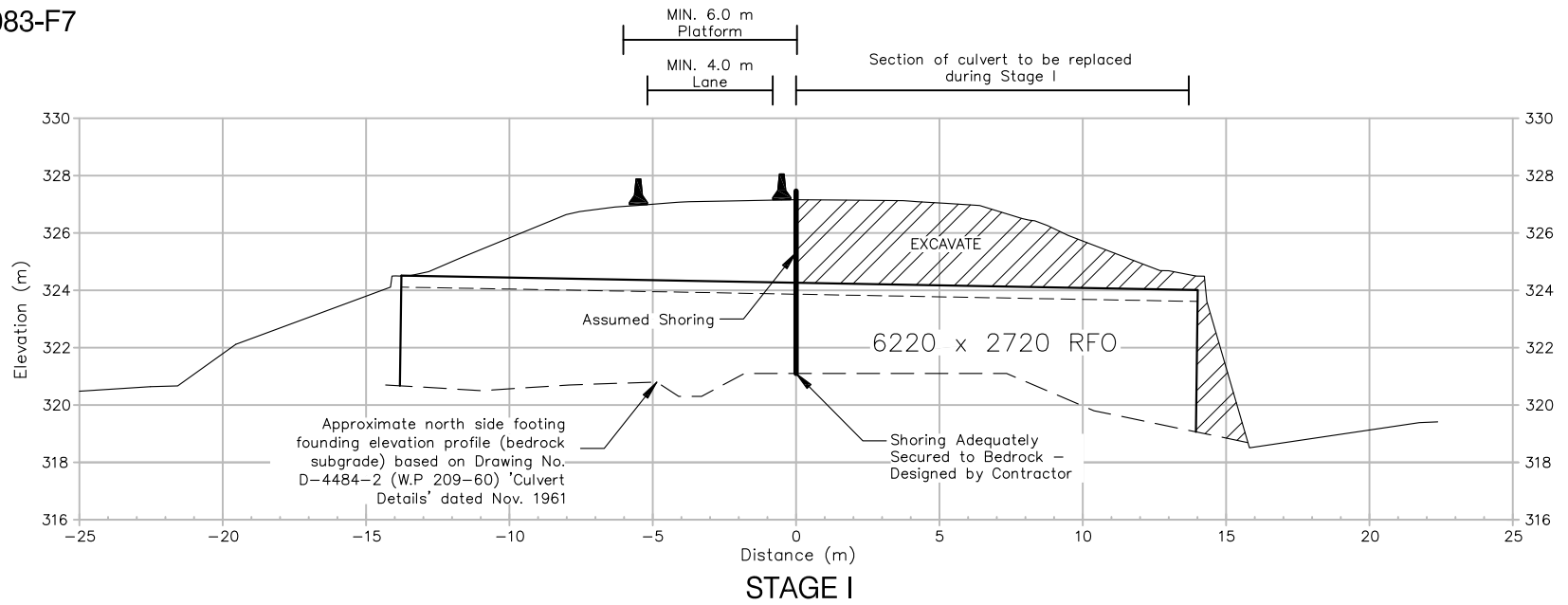
## METRIC

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



Highway 60, Township of Franklin - Culvert at Station 16+310  
Conceptual Shoring Location Plan

FIGURE SK-3



## METRIC

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



Highway 60, Township of Franklin - Culvert at Station 16+310  
Conceptual Shoring Cross-Section

FIGURE SK-4



