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

Little Mollie Creek Culvert
Twp. of Vrooman
Site No. 47-401/C

Highway 560, 6.5 km East of Junction with Highway 144
GWP 5263-10-00

FINAL FOUNDATION INVESTIGATION AND DESIGN REPORT

Date: December 06, 2013
Ref. Nº: 13/05/13073-F8

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Final Foundation Investigation and Design Report

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1 INTRODUCTION

LVM | MERLEX has been retained by AECOM Canada Ltd., on behalf of the Ministry of Transportation of Ontario (MTO), to carry out a foundation investigation at the site of an existing centerline culvert. The site is located on Highway 560, some 6.5 km East of Highway 144 junction, in the Township of Vrooman.

The foundation investigation location was specified by the MTO in the Terms of Reference for work under Agreement No. 5012-E-0025. The terms of reference for the scope of work are outlined in LVM | MERLEX's Proposal P-13-022, dated February, 2013. The purpose of this investigation was to determine the subsurface conditions in the area of the 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 site of this foundation investigation is located on Highway 560 some 6.5 km East of the Junction With Highway 144 in the Township of Vrooman. The local topography at the site is a low wetland to the left and right of the embankment. The existing highway embankment currently supports two undivided lanes of highway, running in a west to east direction. The existing highway, at the culvert location, is constructed on a granular fill embankment some 3.6 m in height, with centerline elevation of 386.1 m at the culvert location. The culvert at this location has been described, in the RFP, as a 3.6 m diameter CSP culvert, some 21.0 m in length, measured along the invert. However, field measurements indicate that the culvert is a SPCSPA of nominal dimension 3600x2800 mm. Flow through the culvert is from south to north (right to left) (see Photo Essay, Appendix 4).

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

2.1 SITE PHYSIOGRAPHY AND SURFICIAL GEOLOGY

This project is located in the Geomorphic Sub-province known as the Eastern Sandy Uplands. The topography on this section of Highway 560 is generally flat. Significant layers of earth overlay the bedrock. Organic terrain was also observed. Within the project area native overburden consists primarily of shallow sand deposits.

Bedrock in the area, as indicated on OGS Map 2506, is of the Early Precambrian period consisting of Felsic Igneous and Metamorphic rocks. At the location of this culvert foundation investigation, the bedrock generally comprises of granitic rocks, syenite, pegmatite, unsubdivided migmatite.

3 INVESTIGATION PROCEDURES

The field work for this investigation was carried out on July 3rd, 2013 during which time four (4) sampled boreholes were advanced. Bedrock coring was undertaken at two borehole locations on August 15th and 16th, 2013. Two (2) boreholes were advanced through the embankment at the location of the culvert, and one borehole was advanced at each the inlet and outlet ends of the culvert.

The field investigation was carried out using a Bombardier mounted CME drilling rig equipped with hollow stem augers, standard augers, 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 mounted in a trip (automatic) hammer. The number of blows per 300 mm penetration was recorded as the “N” value. When shallow auger refusal was encountered at the boreholes located at the culvert ends, NQ size diamond coring equipment was used to determine the nature of shallow refusal. 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. Standpipes were installed in select open boreholes prior to backfilling. 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. 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 surface treatment.

The field work for this investigation was under the full time direction of a senior member of our engineering staff, who was responsible for locating the boreholes, clearing the borehole locations of underground services, in-situ sampling and testing operations, logging of the boreholes, labeling and preparation of samples for transport to our North Bay laboratory, plus overall drill supervision. All samples received a visual confirmatory inspection in our laboratory. Laboratory testing of select samples included routine testing for natural moisture content determination and particle size analysis. The results of the laboratory testing are presented on the individual Record of Borehole Sheets (Appendix 2), with a summary of results presented on the laboratory sheets in Appendix 3 (Figures Nos. L-1 to L-3).

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 exp. Services. The benchmark used at the culvert at Station 10+000 was described as a nail and washer in the south face of Hydro Pole at Station 10+016.5, 15.5 m left of centerline (see Drawing No. 2, Appendix 3).

4 SUBSURFACE CONDITIONS

Details of the subsurface conditions revealed by the investigation program are presented on the enclosed Record 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 10+000, TWP OF VROOMAN

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, four (4) sampled boreholes were put down at this site, with Borehole Nos. 1 and 4 advanced at the culvert ends (inlet (right/south) and outlet (left/north), respectively), and Borehole Nos. 2 and 3 advanced through the embankment. At the time of the subsurface investigation, the ground surface elevations at Boreholes Nos. 1 to 4 were recorded at 384.3, 386.0, 385.8, and 384.3 m, respectively.

4.1.1 Pavement Structure

Borehole Nos. 2 and 3 were advanced on the shoulder where a pavement structure consisting of surface treatment underlain by some 125 to 150 mm of crushed gravel was penetrated.

4.1.2 Surficial Organics/Peat

At ground surface, at Borehole No. 1, a layer of surficial organics some 100 mm thick was penetrated. At ground surface, at Borehole No. 4, a 1.2 m thick layer of black fine fibrous peat was penetrated. The natural moisture content measured on samples of this peat deposit was in the order of 167 to 272%. The peat deposit was encountered to a depth of 1.2 m below grade at Borehole No. 4 (elevation 383.1 m).

4.1.3 Embankment Fill

Underlying the pavement structure, at Borehole Nos. 2 and 3, a layer of granular fill consisting of brown sand trace to some silt trace to some gravel was penetrated. The natural moisture content measured on samples of this deposit was in the order of 3 to 17%. Gradation analyses were carried out on four (4) samples of this deposit, the results of which indicated 4 to 13% gravel size particles, 76 to 88% sand size particles, and 7 to 11% silt and clay size particles (Figure No. L-1, Appendix 3). Based on SPT 'N' values of 3 to 30 blows per 300 mm penetration, the compactness of this deposit was described as very loose to compact, generally loose. This deposit was encountered to a depth of 3.6 m at Borehole Nos. 2 and 3 (elevations 382.4 and 382.2 m, respectively).

4.1.4 Sand

Underlying the thin layer of surficial organics at Borehole No. 1, underlying the peat at Borehole No. 4, and underlying the embankment fill at Borehole Nos. 2 and 3, a deposit of grey sand some to with silt trace to some gravel was penetrated. Occasional cobbles/boulders were encountered in the lower reaches of this deposit at Borehole No. 4. The natural moisture content measured on samples of this deposit was in the order of 8 to 23%. Gradation analyses were carried out on two (2) samples of this deposit, the results of which indicated 0 to 14% gravel size particles, 65 to 86% sand size particles, and 14 to 21% silt and clay size particles (Figure No. L-2, Appendix 3). Based on SPT 'N' values of 0 (static weight of hammer) to greater than 100 blows per 300 mm penetration, this deposit was described as very loose to very dense, generally very dense. Auger refusal was encountered in this deposit at depths of 1.7, 5.5, 4.0, and 2.5 m below grade at Borehole Nos. 1 to 4, respectively (elevations 382.6, 380.5, 381.8, and 381.8 m, respectively). Diamond core drilling was undertaken past auger refusal at Borehole Nos. 1 and 4, to confirm the nature of the refusal material. Sands with cobbles and boulders were encountered between depths of 1.7 to 2.3 m below grade (elevation 382.6 to 382.0 m) at Borehole No. 1, before encountering the bedrock surface.

4.1.5 Bedrock

Underlying the above described sands at Borehole Nos. 1 and 4, bedrock was proven by diamond core drilling. The bedrock was described as grey granitic bedrock. Based on RQD values of 76 to 100% the bedrock was described as good to excellent quality. Sampling in the bedrock was terminated at depths of 5.3 and 5.5 m below grade at Borehole Nos. 1 and 4, respectively (elevations 379.1 and 378.8 m, respectively). It should be noted that, when encountered, the underlying bedrock surfaces in this area are very erratic in nature, varying substantially in elevation over short horizontal distances.

4.2 GROUNDWATER DATA

At the time of this investigation, the water level at the culvert outlet was measured at elevation 383.8 m.

Measurements of the groundwater and cave-in levels were undertaken, where possible, in the open boreholes during the advance of the individual borings and upon completion. Standpipes were installed in Borehole Nos. 1, 2 and 4, to obtain post completion water levels. These levels are recorded on the individual Record of Borehole Log Sheets (Appendix 2). The water levels in Borehole Nos. 1, 2, and 4, were measured at elevation 383.9 m. These water levels appeared stable at the time of drilling.

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 the existing culvert on Highway 560 at Little Mollie Creek, as identified in the RFP.

The existing culvert, located at Station 10+000 in the Township of Vrooman, has been described as a 3.6 m diameter CSP culvert some 21.0 m long, at the invert. However, field measurements indicated a nominal culvert size of 3600x2800 mm and the culvert appears to be a pipe arch. The culvert is currently being supported with an interior shoring system (see Photos Appendix 4). The culvert has been independently measured by the structural team and they have drawn a similar conclusion that the culvert is a SPCSPA of nominal dimension 3600x2800 mm.

The existing highway embankment currently supports two undivided lanes of highway, running in a west to east direction. While flow through the culvert was not observed at the time of this investigation, we understand that the flow through the culvert is from right to left (south to north). Based on data from this foundation investigation, the embankment supporting the existing pavement structure at this site has been constructed using a granular pavement structure overlying essentially granular fill. The native material, underlying the embankment fill, generally consisted of very dense sands over bedrock, at a shallow depth.

The type of culvert (concrete, CSP, or High Density Polyethylene (HDPE)) which will be used to replace the existing culvert is currently unknown. However, it is understood that a 3600x3000 mm concrete box culvert is being proposed for replacement at this detailed design stage. It is understood that the new culvert will be constructed at a similar alignment and skew as the existing culvert. It is understood that there is a historical channel alignment, located directly to the west of this culvert, however, due to environmental concerns this channel should only be used for temporary by-pass purposes. The final vertical alignment of the highway will remain essentially the same.

5.2 FOUNDATION CONSIDERATIONS

The founding native dense/very dense sands overlying shallow bedrock below the existing embankment are considered excellent materials for support of a culvert and for a conventional highway embankment of this height. Bearing resistance should not be a major issue provided the natural bearing surface is not excessively disturbed during construction and groundwater is controlled during construction, as discussed in Section 5.6.

Based on the characteristics of the native sand subgrade present below the culvert, the response of the existing embankment, and a founding elevation at or below the existing culvert invert, a factored bearing resistance at ULS of 650 kPa can be used for a closed culvert (i.e. precast concrete frame box culvert with 3.6 m span or SPCSPA culvert). In consideration of the width of the culvert, limited depth of overburden, and response of the existing embankment, a geotechnical reaction at SLS of 350 kPa can be used for design, in consideration of less than 25 mm settlement.

If open culverts (i.e. concrete frame open culverts, with 1 m wide wall footings, or pipe arch culverts on 1 m wide footings) are considered, the depth of footing below stream bed will probably be in the order of 1.5 m, or greater, for scour protection. At this depth the strip footings will be at or very close to the depth that bedrock was proven and depth auger refusal was met. As such, all 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 bedrock is of excellent quality, based on RQD data. As such, a bearing resistance at ULS of 1500 kPa is appropriate with a minimum footing width of 600 mm. Since bedrock is essentially an unyielding subgrade, a geotechnical reaction at SLS does not apply.

Considering the 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 must 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 300 mm and then downwards on a 45° outward angle to the bedrock surface.

5.2.1 Slope Stability

The maximum height of fill above surrounding grade of the embankment at this location is some 3.6 m above the stream bed at the culvert locations. 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 existing embankment slopes of 2.0H:1.0V in granular fill. For the purposes of these analyses, the materials were modeled using the following parameters;

MATERTIAL TYPE	UNIT WEIGHT (kN/m^3)	FRICTION ANGLE (°)
Embankment Fill	19	29
Sand	19	32
Rock Protection	18.5	43

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 1.5 (see Figure No. S-1, Appendix 5). Lower factors of safety will occur during excavation and backfilling as discussed in Section 5.5. Short term stability should not be an issue if construction is carried out as described herein. The long term stability of the new embankment will not be an issue provided it is properly constructed.

5.3 CULVERT DESIGN, BEDDING, AND EMBEDMENT

The embankment consists of granular fills. The results of this investigation indicate that, below the culvert inverts, the native soils at Borehole Nos. 3 and 4 consist of dense sands overlying bedrock at a shallow depth. Based on a review of the condition of the pavement surface, at the culvert location, it appears that the embankment has 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 increase in embankment load, no appreciable settlement of the embankment is anticipated. In addition, bedrock, which was encountered at relatively shallow depths, is considered non-yielding subgrade. As such, installing the culvert on a camber will not be required at this site.

5.3.1 Rigid Concrete Culvert

A concrete pipe or concrete box has been 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 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.

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 new culvert width and beyond by 2 m to either side. Clay seals are generally used where significant head differences exist between the inlet and outlet of the culvert, or where a relatively impermeable subgrade is present, to prevent flow through, and piping in, the embankment. Clay seals are not considered necessary considering the anticipated water levels and flow at this culvert location.

5.3.2 Flexible Steel 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 maintained at 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 width, 400 mm thick and extend across the width of the culvert and beyond by 2 m to both sides.

5.4 CULVERT INSTALLATION AND CONSTRUCTION STAGING CONSIDERATIONS

The invert elevation of the existing culvert is at 382.5 m, with the top of the embankment at elevation 386.1 m at centerline. As such, the embankment at this location is some 3.6 m in height above the existing culvert invert at the centerline. Therefore, a minimum 4.5 m deep excavation (i.e. to elevation 381.6 m) will be required in consideration of countersinking a box culvert some 300 mm, a 300 mm thick concrete culvert bottom, and a 300 mm thick layer of bedding material. This is some 200 mm below the bedrock level encountered during the investigation (i.e. 382.0 m at Borehole No. 1). It should be noted that, when encountered, the underlying bedrock surfaces in this area are very erratic in nature, varying substantially in elevation over short horizontal distances. As such, to minimize potential bedrock removal, it is suggested that the culvert invert be established as high as possible. A minimum thickness of 300 mm of bedding/embedment material must be provided over bedrock subgrade.

The present platform width at this location is some 12 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 a temporary detour is constructed at this location. It is understood that a temporary detour is not being considered for use during culvert replacement. As such, consideration should be given to constructing a vertical wall along centerline for use as a protection system during culvert replacement.

5.4.1 Protection System

As noted above, a vertical wall installed along centreline for use as a temporary protection system can be used during culvert replacement to maintain an active lane of traffic. The installation of a protection system for use in the culvert replacement operation will require penetration through some 3.6 m of embankment fill. The embankment fill is generally underlain by dense sands with cobbles and boulders. One possible method of constructing a temporary vertical wall for roadway protection along the centreline of the highway alignment would be to drive steel sheet piles through the embankment fill into the native material. If a cobble/small boulder size rock is encountered during driving of a sheet section, the individual section could be left high and the cobble/small boulder removed during excavation to allow continued driving. Due to the relatively shallow depth of bedrock, and very dense compactness of the native sand deposits, at this culvert location, there may not be sufficient penetration to provide toe resistance. As such, a lower waler support may have to be incorporated in the shoring design. The sheet pile design should be carried out by a structural engineer with experience designing sheet pile walls. Conceptual shoring locations are illustrated on Figure Nos. SK-4 and SK-5, Appendix 5.

See Table A, Appendix 5, for advantages and disadvantages for the different type of protection system considered for this site.

Considering the limited depth of excavation, and provided a sheet pile of sufficiently robust section 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):

$R = \sigma z' \, As \, Ls \, \alpha g$ Where: $\sigma z'$ = effective vertical stress at the midpoint of the load carrying length

As = effective unit surface area of the anchor

Ls = effective embedment length of the anchor

αg = 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.

As noted, bedrock was encountered at relatively shallow depths at this culvert location, which will affect the toe resistance of driven sheet piles. As such, consideration can be given to using a micropile system, with micropiles drilled into bedrock, however this shoring system is generally more costly. Alternatively, a caisson wall or soldier piles with lagging can be used. Caissons and or soldier piles should be predrilled and advanced into the bedrock to provide sufficient toe resistance.

Considering the cohesionless nature of the embankment fills (granular pavement structure over essentially 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 $0.65 * Ka * \gamma * H$, where:

Ka = active earth pressure,

γ = unit weight, and

H = height of wall above the base of excavation.

The protection system can be designed using the lateral earth pressure parameters provided in Section 5.5 Lateral Earth Pressures.

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	SAND
Unit Weight (kN/m^3)	22.8	21.2	19	19
Angle of Internal Friction	34°	31°	30	32
Coefficient of Active Earth Pressure (Ka)	0.28	0.32	0.33	0.31
Coefficient of Passive Earth Pressure (Kp)	3.54	3.12	3.00	3.25
Coefficient of Earth Pressure at Rest (Ko)	0.44	0.48	0.50	0.47

For rigid structures, such as a precast concrete culvert, deflection cannot occur, as such the "at-rest" condition (Ko) applies. For flexible structures, such as CSP/HDPE culverts, deflection can occur, as such the "active" condition (Ka) 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. Final (permanent) embankment side slopes in granular fill should be established to match the existing side slopes, with an angle of 2.0H:1.0V.

It should be noted that cobbles and boulders were encountered during this investigation. Additionally, the native deposits were generally dense. As such, the contractor must be prepared to use equipment of sufficient capacity to excavate these materials.

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 383.8 m at the outlet at the time of this investigation and excavations to an approximate elevation 381.9 m would 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. To provide a stable working surface the water level must be lowered to below the base of excavation. When wet, the subgrade can become easily disturbed, and can lose a significant portion of its native bearing capacity. A sand (metre) bag cofferdam, earth fill cofferdam, aquadam, or possibly temporary sheet pile type cofferdam can also be considered for controlling stream flow depending upon anticipated flow at time of construction. Considering the shallow bedrock at this site, a sand bag, aquadam, or earth fill cofferdam are recommended at this site. By-pass pumping through a temporary culvert can be carried out to divert the stream flow at the time of construction.

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 foundation construction cannot be stressed enough.

5.6.1 Bedrock Excavation

Bedrock was encountered at a shallow depth below the existing culvert invert. Depending on the invert of the new culvert, bedrock may be within the anticipated depth of excavation. Therefore bedrock excavation and/or blasting operations may be required.

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

5.7 CONSTRUCTION CONCERNS

Considering the nature of the granular fill embankment, no major construction concerns are anticipated if construction is carried out in general conformance with the above discussion. It is again noted that cobbles and boulders were encountered during this investigation and the native deposits were generally dense. As such, the contractor must be prepared to use equipment of sufficient capacity to penetrate/excavate these materials. In addition, the presence of shallow bedrock will affect the design the temporary protection system and dewatering system.

Appendix 1 Key Plan

Drawing No. 1 Key Plan

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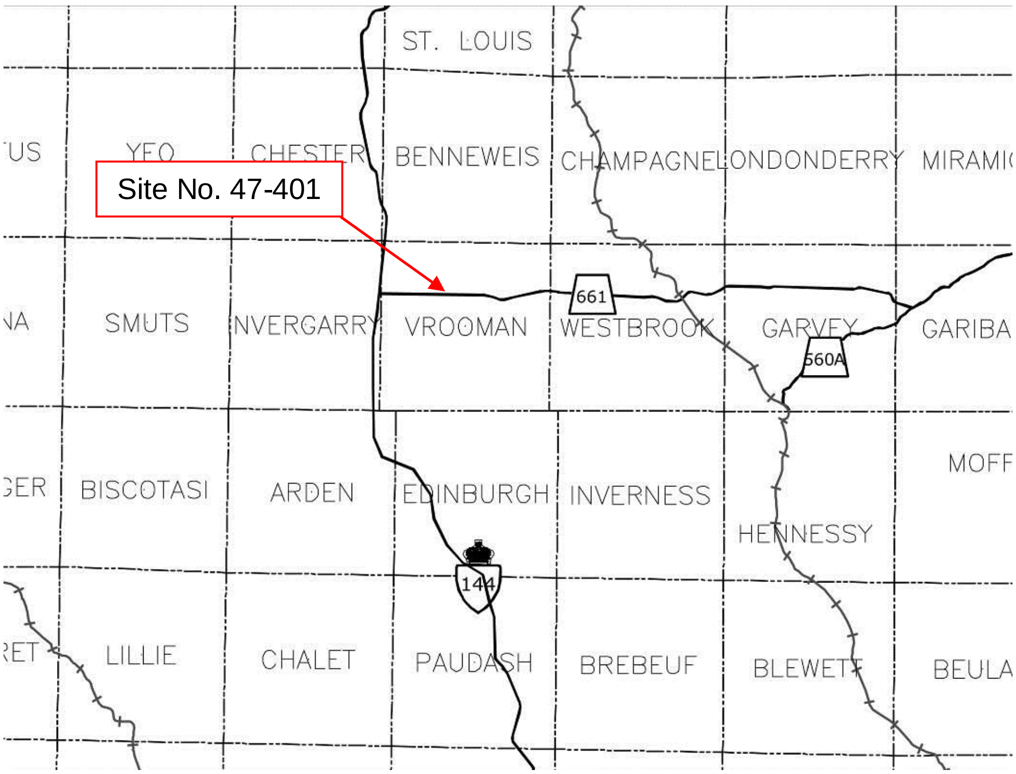
KEY PLAN

Drawing No. 1

NOT TO SCALE



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Appendix 2 Subsurface Data

Enclosure No. 1	List of Abbreviations and Symbols
Enclosure Nos. 2 to 5	Record of Borehole Sheet

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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

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) *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.

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/or boulders frequency is an estimate based on drill response and field observations:

Occasional	Obstructions encountered in borehole, however advance is not severely impeded
Numerous	Obstructions appear essentially continuous over drilled length

5. LABORATORY TESTS

P Standard Proctor Test
A Atterberg Limit Test
GS Grain Size Analysis
H Hydrometer Analysis
C Consolidation

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.

ENCLOSURE NO.:2 (Pg. 1 of 1)

RECORD OF BOREHOLE NO. 1

METRIC

REFERENCE 13/05/13073-F8 DATUM Geodetic LOCATION N 5259319.7 E 247453.0 - Township of Vrooman ORIGINATED BY JL

PROJECT GWP 5263-10-00, Hwy 560 - Sta 10+000 Twp of Vrooman BOREHOLE TYPE Track Mounted CME 45B - Hollow Stem Augers COMPILED BY AT

CLIENT AECOM Inc. DATE (Started) 2013 July 3 DATE (Completed) 2013 July 3 TIME (Completed) CHECKED BY MAM

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA (SI CL)
384.3	Ground Surface					
0.0	100 mm surficial organics SAND - grey sand trace to some silt trace gravel (very loose/loose)	1	SS	WH	o	
		2	SS	8	o	0 86 (14)
382.6		3	SS	25/50 mm	o	
1.7	Auger Refusal Advanced NW Casing to 1.7 m depth, start coring Cobbles/boulders	4	RC			
382.0 2.3	BEDROCK - grey granitic bedrock excellent quality	5	RC	Rec= 100% RQD= 100%		
		6	RC	Rec= 100% RQD= 90%		
379.1 5.2	End of Borehole					

ELEVATION SCALE: 384, 383, 382, 381, 380

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100
SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE; 20 40 60 80 100
PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60
UNIT WEIGHT γ kN/m³

COMMENTS

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

+3, ×3 : Numbers on right refer to Sensitivity. Numbers on left refer to values greater than 120 kPa

○ 3% STRAIN AT FAILURE

WATER LEVEL RECORDS

Date (dd/mm/yy)/Time	Water Depth (m)	Cave In (m)
1) 13/7/3 12:00:00 PM	0.9	1.7
2) 13/7/3 12:20:00 PM	0.4	1.5
3)	-	-

MEL-GEO 13073-F8 - BOREHOL LOGS.GPJ MEL-GEO.GDT 13/10/1

ENCLOSURE NO.:3 (Pg. 1 of 1)

METRIC

RECORD OF BOREHOLE NO. 2

LVM

REFERENCE 13/05/13073-F8	DATUM Geodetic	LOCATION N 5259323.5 E 247463.9 - Township of Vrooman	ORIGINATED BY JL
PROJECT GWP 5263-10-00, Hwy 560 - Sta 10+000 Twp of Vrooman		BOREHOLE TYPE Track Mounted CME 45B - Hollow Stem Augers	COMPILED BY AT
CLIENT AECOM Inc.		DATE (Started) 2013 July 3 / DATE (Completed) 2013 July 3 / TIME (Completed)	CHECKED BY MAM

ELEV DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA (SI CL)
386.0	Ground Surface								
0.0	25 mm Asphalt 150 mm Crushed Gravel FILL - brown sand trace to some silt trace to some gravel (compact)	Fill	1	SS	23				
			2	SS	17		385		13 76 (11)
			3	SS	7	▽ (1.8 m)			
						▽▼ (2.1 m)	384		
			4	SS	8	Cave in (2.6 m)			
			5	SS	3		383		6 87 (7)
382.4 3.6	SAND - grey sand with silt some gravel (very dense)	Sand	6	SS	58		382		14 65 (21)
			7	SS	50/125 mm		381		
380.5 5.5	Auger Refusal End of Borehole								

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE; 20 40 60 80 100

PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60

UNIT WEIGHT γ kN/m³

COMMENTS

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

+3, ×3: Numbers on right refer to Sensitivity. Numbers on left refer to values greater than 120 kPa

○ 3% STRAIN AT FAILURE

WATER LEVEL RECORDS

Date (dd/mm/yy)/Time	Water Depth (m)	Cave In (m)
1) 13/7/3 10:30:00 AM	1.8 ▽	2.6
2) 13/7/3 4:30:00 PM	2.1 ▽	-
3) 13/7/4 9:00:00 AM	2.1 ▼	-

MEL-GEO 13073-F8 - BOREHOL LOGS.GPJ MEL-GEO.GDT 13/10/1

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

ENCLOSURE NO.:4 (Pg. 1 of 1)

RECORD OF BOREHOLE NO. 3

METRIC

LVM

REFERENCE 13/05/13073-F8 DATUM Geodetic LOCATION N 5259332.9 E 247460.8 - Township of Vrooman ORIGINATED BY JL

PROJECT GWP 5263-10-00, Hwy 560 - Sta 10+000 Twp of Vrooman BOREHOLE TYPE Track Mounted CME 45B - Hollow Stem Augers COMPILED BY AT

CLIENT AECOM Inc. DATE (Started) 2013 July 3 DATE (Completed) 2013 July 3 TIME (Completed) CHECKED BY MAM

ELEV DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA (SI CL)
385.8	Ground Surface							
0.0	25 mm Asphalt 125 mm Crushed Gravel FILL - brown sand trace to some silt trace gravel (loose/compact)	Fill	1	SS	30			
			2	SS	8		385	
			3	SS	6	Cave in	384	5 84 (11)
			4	SS	4		383	
			5	SS	5			4 88 (8)
382.2 3.6	SAND - grey sand trace to some silt trace gravel	Sand	6	SS	50/75 mm		382	
381.8 4.0	Auger Refusal End of Borehole							

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE — 20 40 60 80 100

PLASTIC LIMIT W_P — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L; WATER CONTENT (%) 20 40 60

UNIT WEIGHT γ kN/m³

COMMENTS

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

+3, ×3: Numbers on right refer to Sensitivity. Numbers on left refer to values greater than 120 kPa

○ 3% STRAIN AT FAILURE

WATER LEVEL RECORDS

Date (dd/mm/yy)/Time	Water Depth (m)	Cave In (m)
1) 13/7/3 2:50:00 PM	DRY	1.5
2)	-	-
3)	-	-

MEL-GEO 13073-F8 - BOREHOL LOGS.GPJ MEL-GEO.GDT 13/10/1

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

ENCLOSURE NO.:5 (Pg. 1 of 1)

METRIC

RECORD OF BOREHOLE NO. 4

LVM

REFERENCE 13/05/13073-F8 DATUM Geodetic LOCATION N 5259336.8 E 247470.6 - Township of Vrooman ORIGINATED BY JL

PROJECT GWP 5263-10-00, Hwy 560 - Sta 10+000 Twp of Vrooman BOREHOLE TYPE Track Mounted CME 45B - Hollow Stem Augers COMPILED BY AT

CLIENT AECOM Inc. DATE (Started) 2013 July 3 DATE (Completed) 2013 July 3 TIME (Completed) CHECKED BY MAM

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	WATER CONTENT (%)	REMARKS
384.3	Ground Surface						
0.0	PEAT - black fine fibrous peat	1	SS	WH	▽ (384)	272	
		2	SS	WH	▽	167	
383.1 1.2	SAND - grey sand some silt trace gravel occasional cobbles/boulders	3	SS	25/50 mm		○ (~14)	
381.8 2.5	Auger Refusal Advanced NW casing to 2.5 m, start coring Bedrock - grey granitic bedrock good quality	4	RC	Rec= 100% RQD= 76%			
		5	RC	Rec= 100% RQD= 82%			
378.8 5.5	End of Borehole						

Elevation scale: 384, 383, 382, 381, 380, 379

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE — 20 40 60 80 100

PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 20 40 60

UNIT WEIGHT γ kN/m³

REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA (SI CL)

COMMENTS

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

+3, ×3 : Numbers on right refer to Sensitivity. Numbers on left refer to values greater than 120 kPa

○ 3% STRAIN AT FAILURE

WATER LEVEL RECORDS

	Date (dd/mm/yy)/Time	Water Depth (m)	Cave In (m)
1)	13/7/3 4:35:00 PM	0.9	2
2)	13/7/3 5:05:00 PM	0.4	1.4
3)		-	-

MEL-GEO 13073-F8 - BOREHOL LOGS.GPJ MEL-GEO.GDT 13/10/1

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

Appendix 3 Borehole Plan and Lab Data

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

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DISTRICT
CONT. No.
GWP No. 5263-10-00

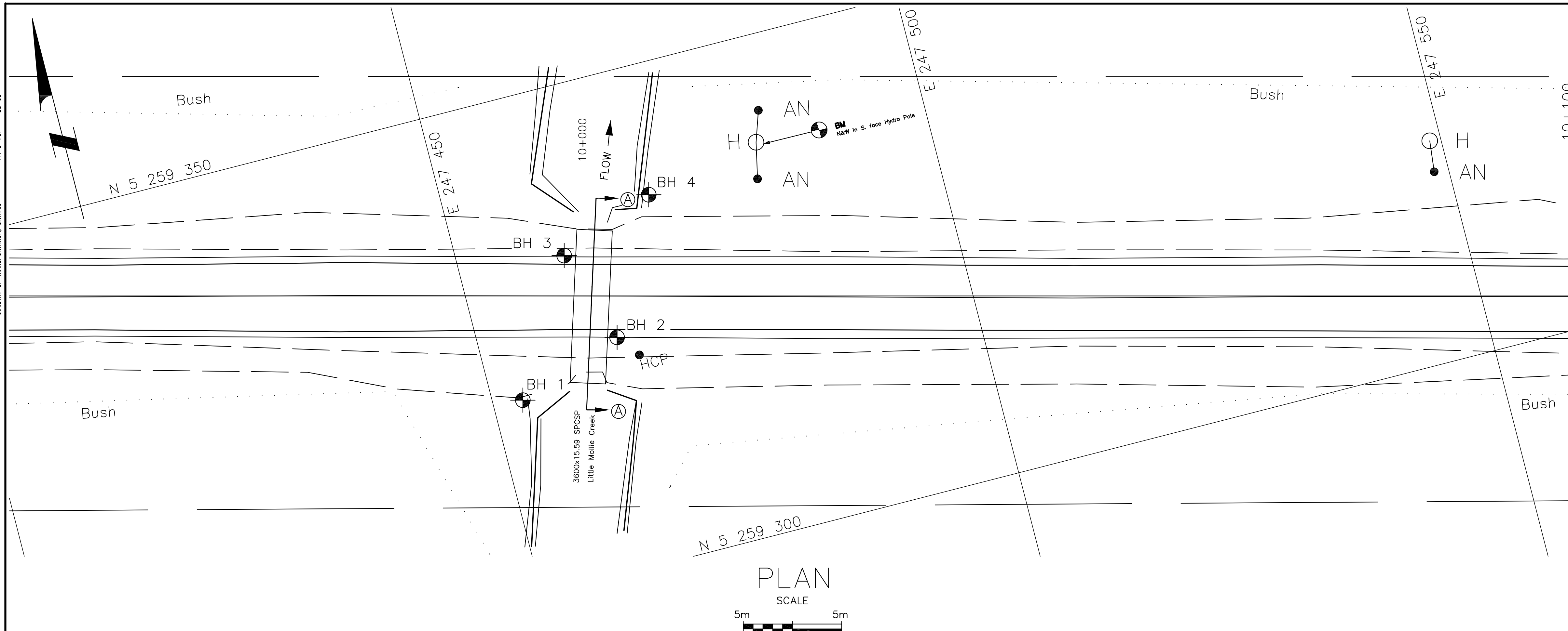
HWY 560
LITTLE MOLLIE CREEK CULVERT
SITE NO. 47-401/C

BOREHOLE LOCATIONS
AND SOIL STRATA

SHEET

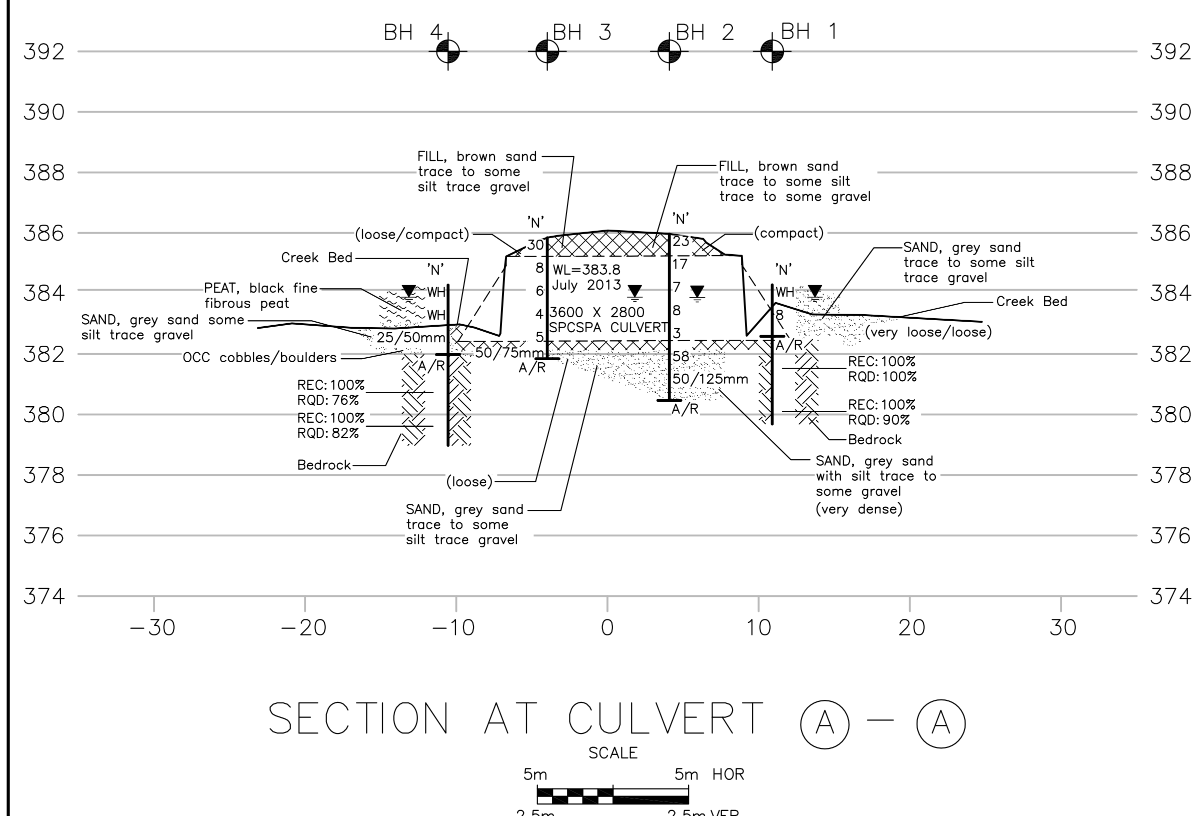
LVM

METRIC



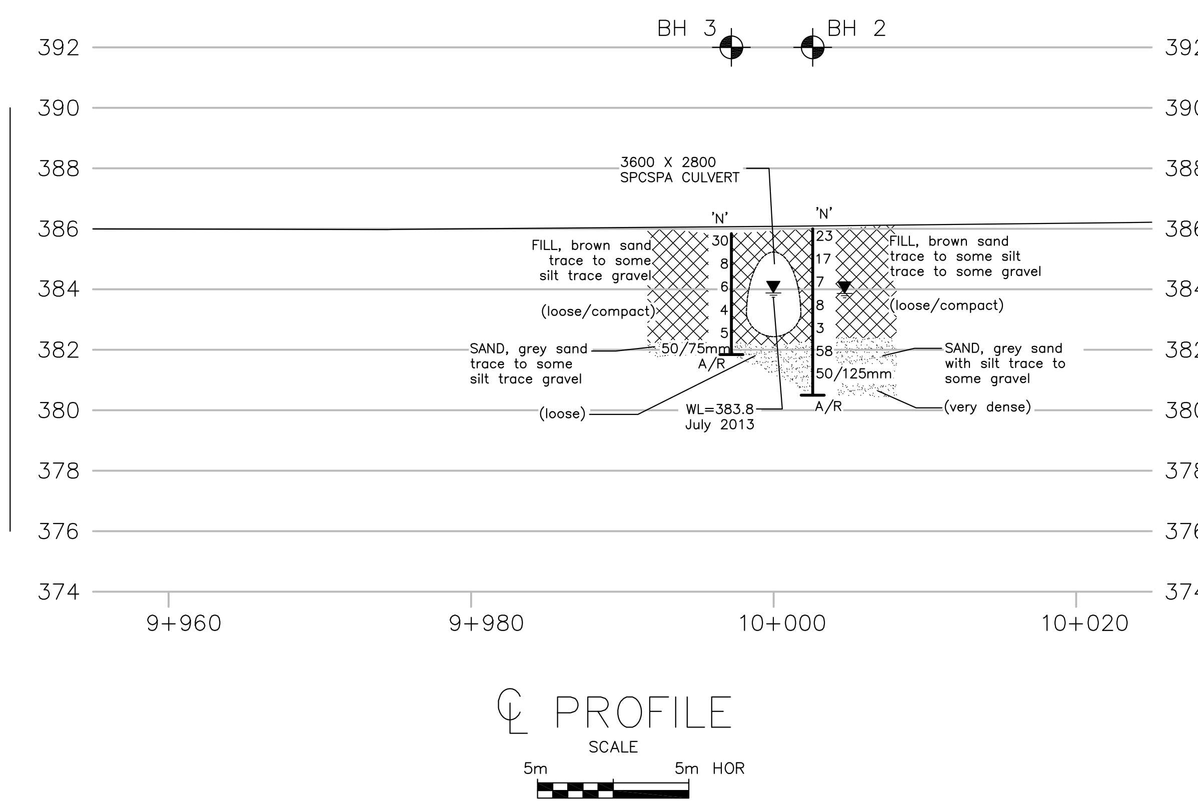
PLAN

SCALE 5m 5m



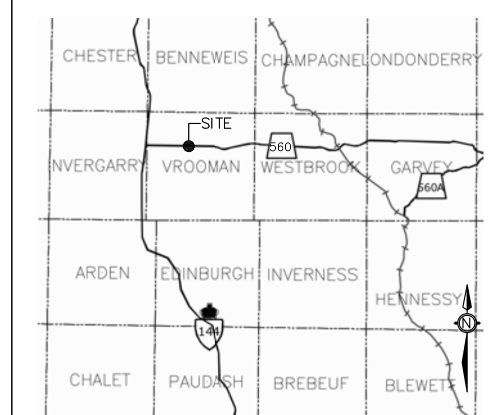
SECTION AT CULVERT (A) – (A)

SCALE 5m 5m HOR; 2.5m 2.5m VER



℄ PROFILE

SCALE 5m 5m HOR; 2.5m 2.5m VER



KEY PLAN

N.T.S.

LEGEND

- Borehole
- 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

BOREHOLE No.	ELEVATION	O/S	NORTHING	EASTING
1	384.3	10.6m Rt	5259319.7	247453.0
2	386.0	4.2m Rt	5259323.5	247463.9
3	385.8	4.1m Lt	5259332.9	247460.8
4	384.3	10.3m Lt	5259336.8	247470.6

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.

GEOCRES No. 41P-53

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

REVISIONS | DESCRIPTION

DESIGN	CHK	CODE	LOAD	DATE DEC/13
DRAWN MCM	CHK AT	SITE 47-401/C	STRUCT	SCHEME	DWG 2

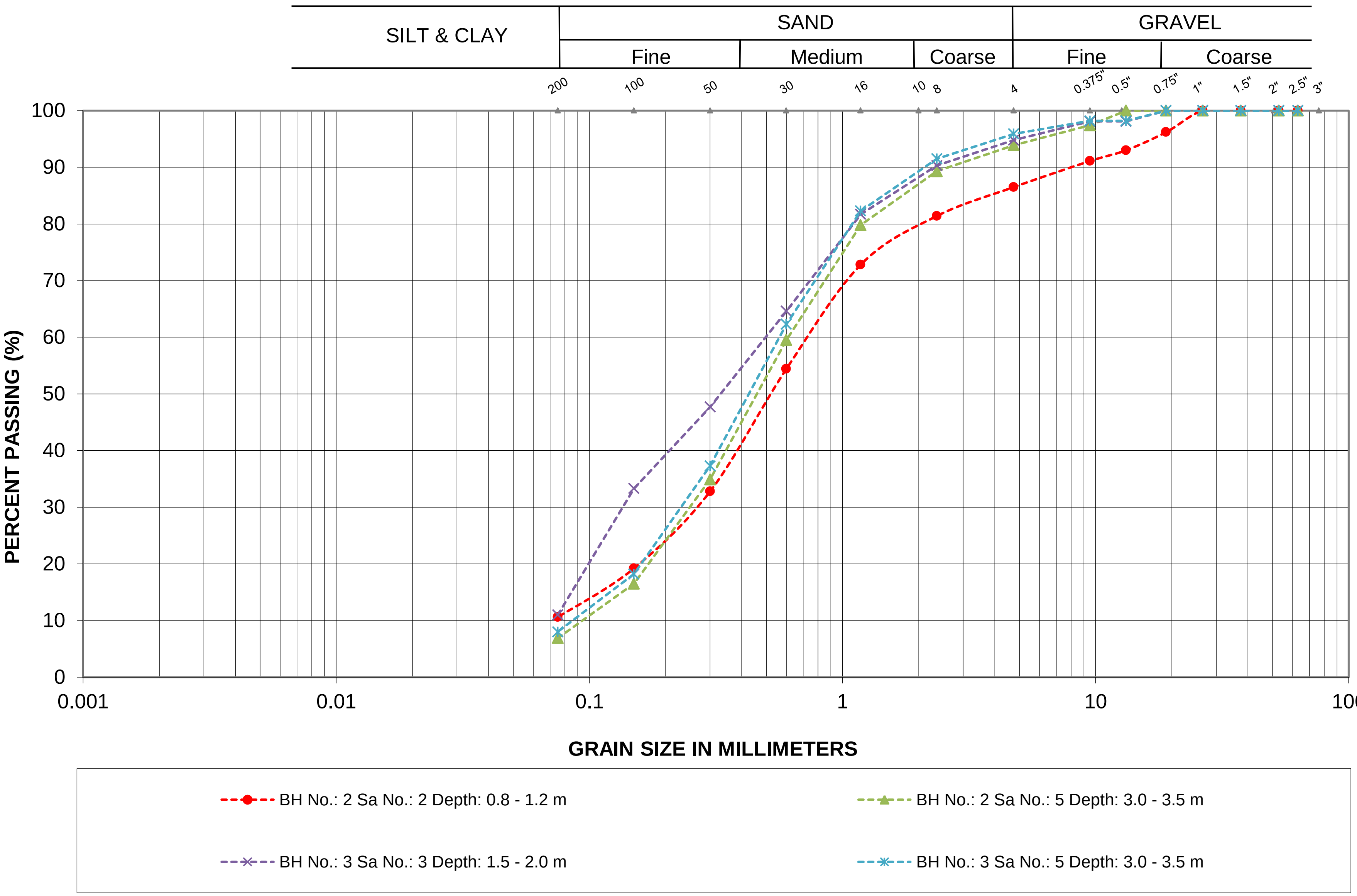
CAD FILE LOCATION AND NAME: ...\2013\13073 - PAVE & FDN, Hwy 65 Various Locations (AECOM)\FOUNDATIONS\FB-Little Mollie GWP 5263-10-00\Report\FINAL\Appendix 3\13073-FB - Borehole Location Plan - Little Mollie Creek Culvert.dwg
MODIFIED: 12/9/2013 2:58:04 PM BY: GRASRY
DATE PLOTTED: 12/9/2013 2:59:37 PM BY: RYAN GRASSER

MINISTRY OF TRANSPORTATION, ONTARIO PR-D-707 88-05

Reference No.: 13/05/13073-F8
Date: December 2013

LVM | MERLEX

GRAIN SIZE ANALYSIS



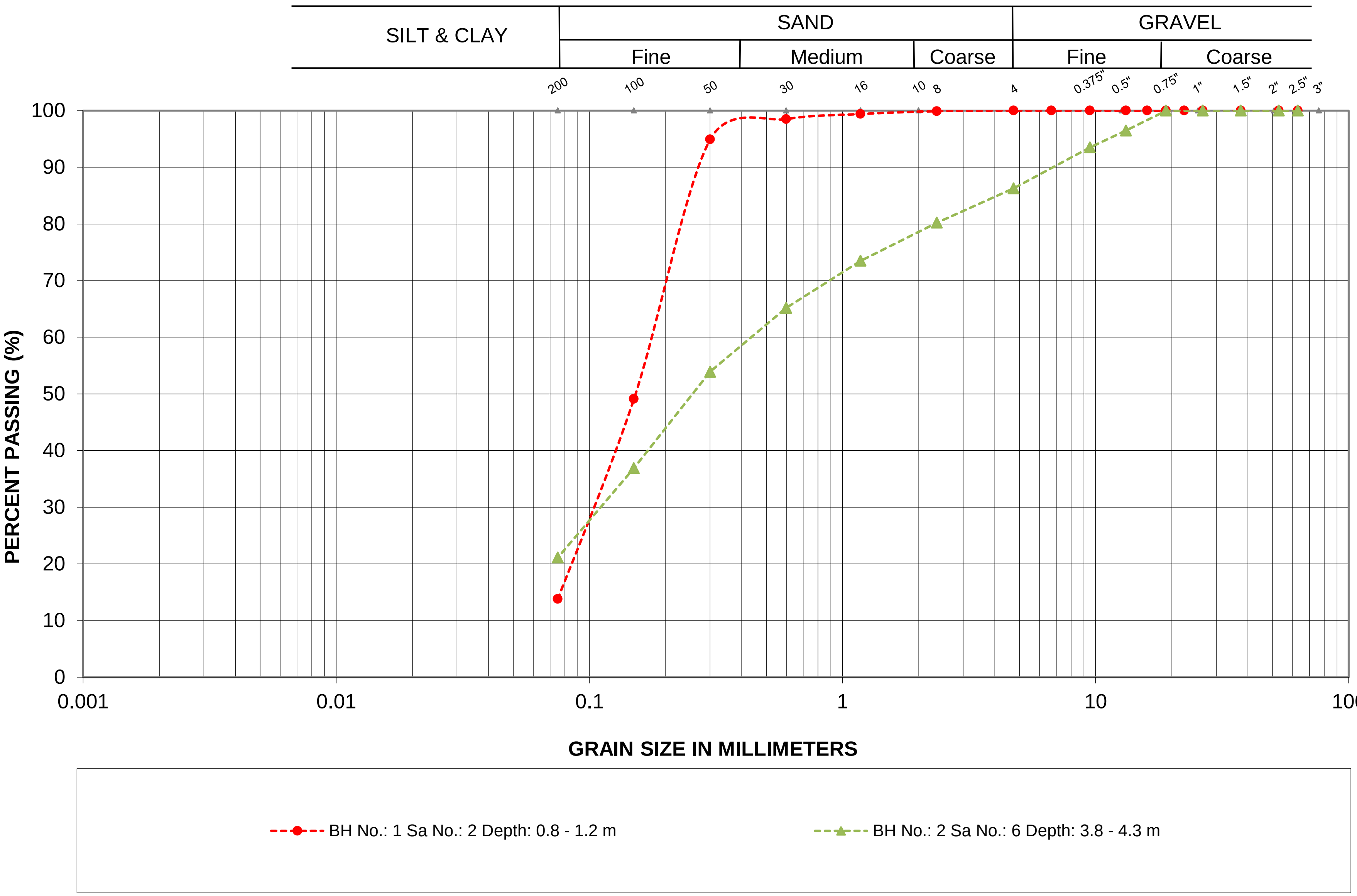
G.W.P.: 5263-10-00
LOCATION: Hwy 560

EMBANKMENT FILL

Reference No.: 13/05/13073-F8
Date: December 2013

LVM | MERLEX

GRAIN SIZE ANALYSIS



G.W.P.: 5263-10-00
LOCATION: Hwy 560

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					20.8				WH			
	2	0.8	0	86	14		23.5				8			
	3	1.5					18.9				25/50 mm			
2	1	0.0					3.5				23			
	2	0.8	13	76	11		7.8				17			
	3	1.5					11.1				7			
	4	2.3					17.1				8			
	5	3.1	6	87	7		16.0				3			
	6	3.8	14	65	21		8.4				58			
	7	4.6					9.0				50/125 mm			
3	1	0.0					7.7				30			
	2	0.8					5.2				8			
	3	1.5	5	84	11		12.3				6			
	4	2.3					15.7				4			
	5	3.1	4	88	8		14.3				5			
	6	3.8					10.8				50/75 mm			
4	1	0.0					272.0				WH			
	2	0.8					167.0				WH			
	3	1.5					13.3				25/50 mm			

Appendix 4 Photo Essay

Enclosure No. 6: Photo Essay

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Existing Embankment at Culvert Location – Looking East	Photo: 1

Culvert Inlet – Looking South	Photo: 2

Project: Hwy 560 – Station 10+000, Twp of Vrooman	Photos Provided By: LVM Date: July 2013

Culvert Outlet – Looking North	Photo: 3

View through culvert – Looking South	Photo: 4

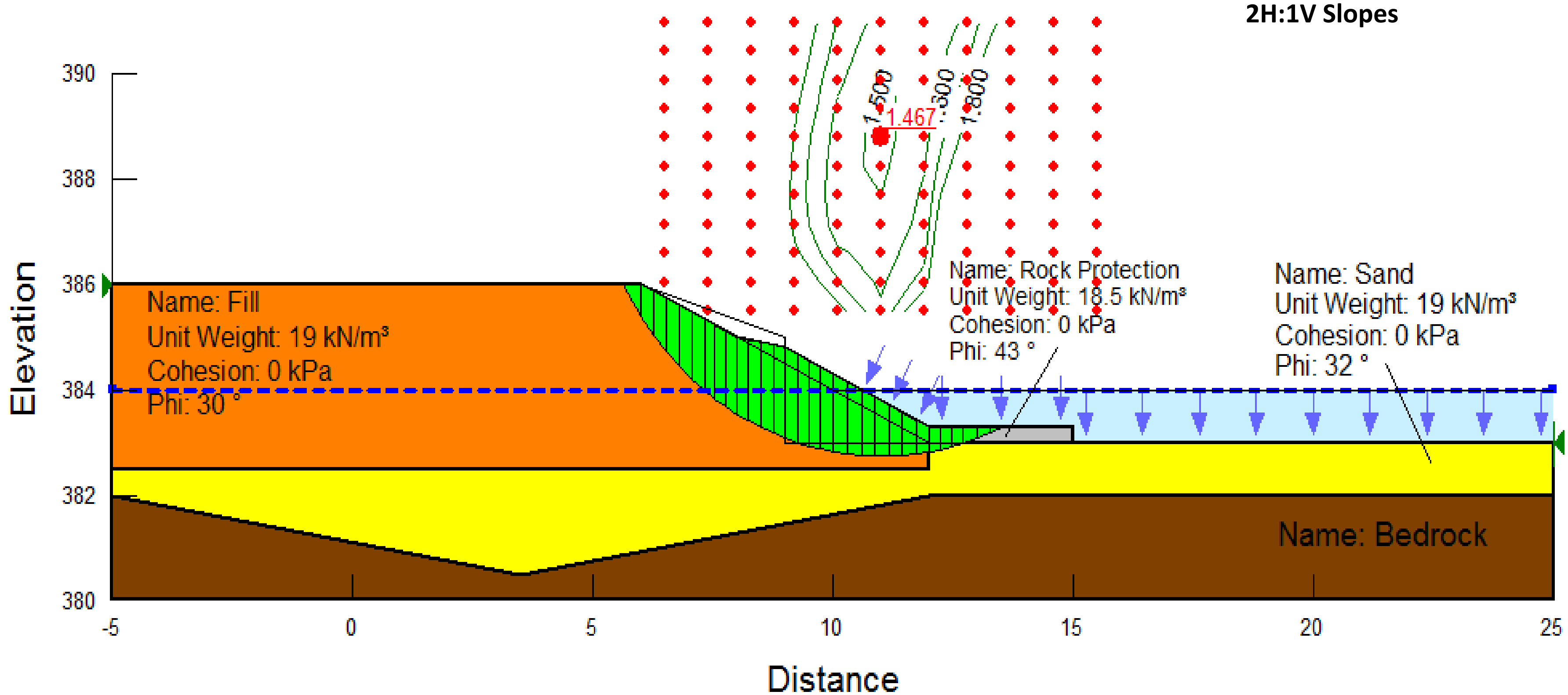
Project: Hwy 560 – Station 10+000, Twp of Vrooman	Photos Provided By: LVM Date: July 2013

Appendix 5 Design Data

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

LVM | MERLEX

Stability Analysis
Embankment Stability
LongTerm Stability
Failure of Native Material
2H:1V Slopes



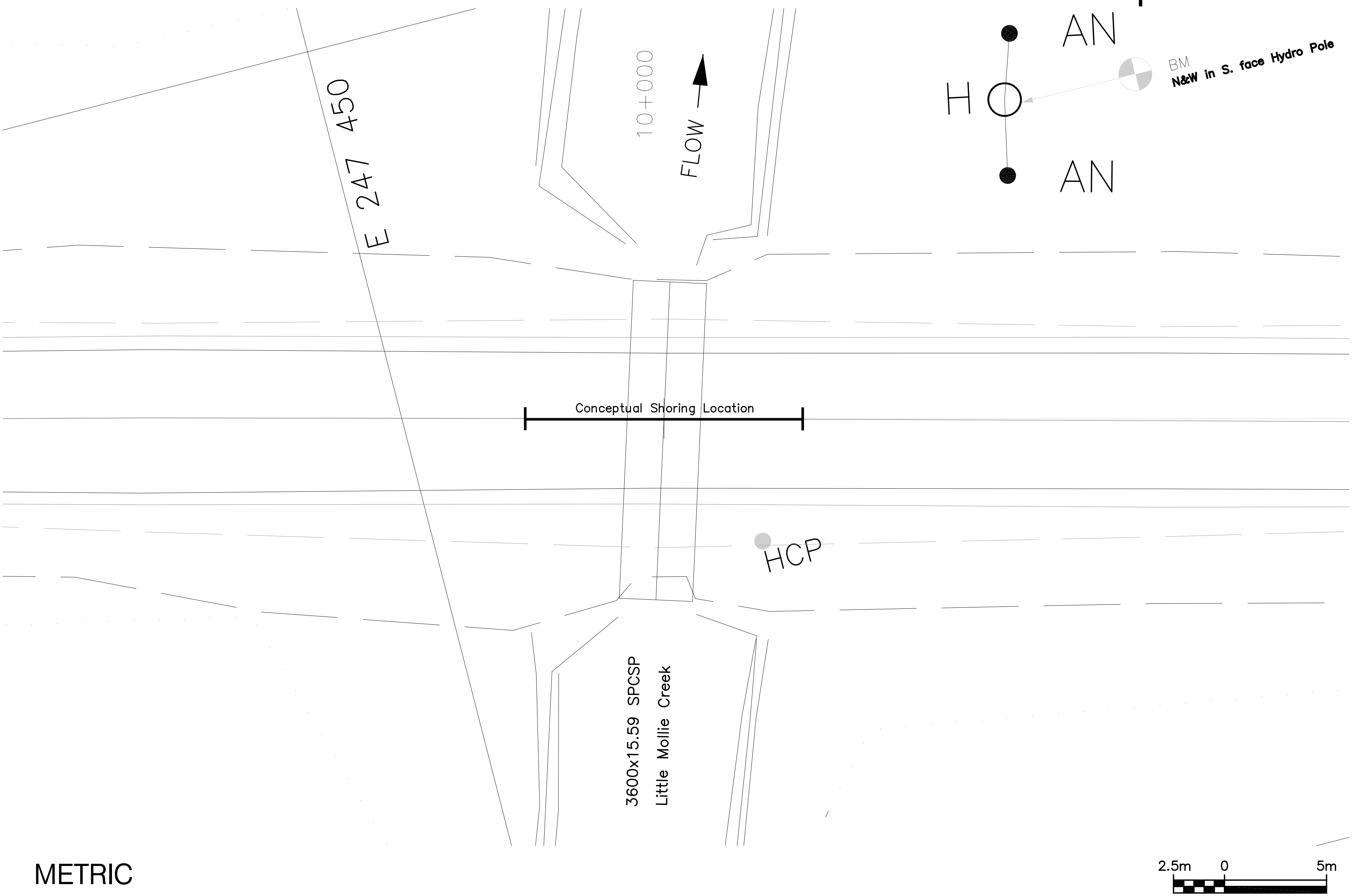
Stability Analysis
Station 10+000
TWP of Vrooman

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	Considered for protection system.	$ 650/m^2
Steel Sheet Piles	5 – 21	-High strength, continuous, -Readily available	-Limited by soil conditions (i.e. obstructions)	Recommended for temporary protection.	$ 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 higher cost.	
Soldier piles	5 – 25	-Easy installation -Readily available -Adaptable to various ground conditions	-Pre-drilling may be required -Possible ground loss	Not considered due to higher cost	$ 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	Not Considered due to higher costs	$ 900/m^2
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	Not Considered due to higher costs	$ 1,200 – 1,500/m^2

REF.: 13073-F8

LVM | MERLEX



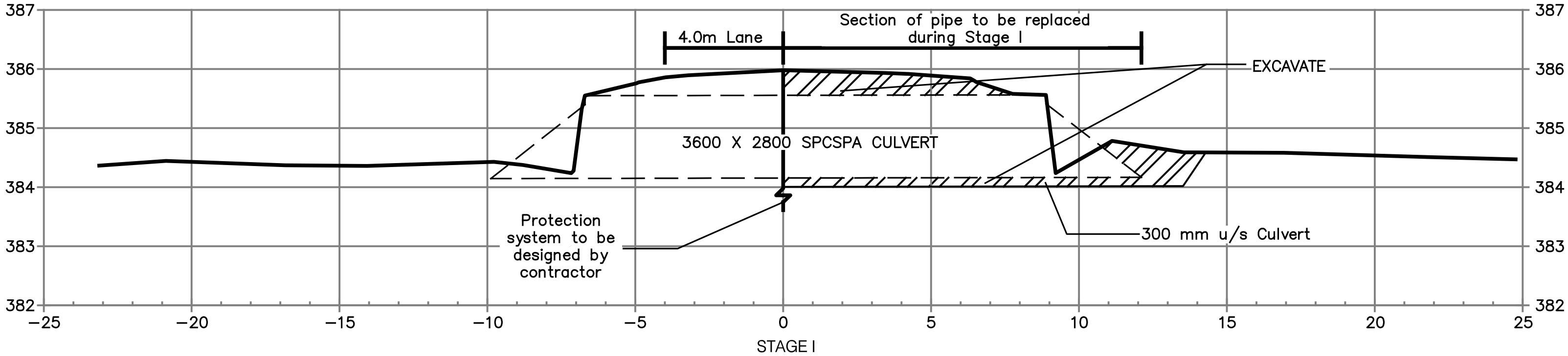
METRIC

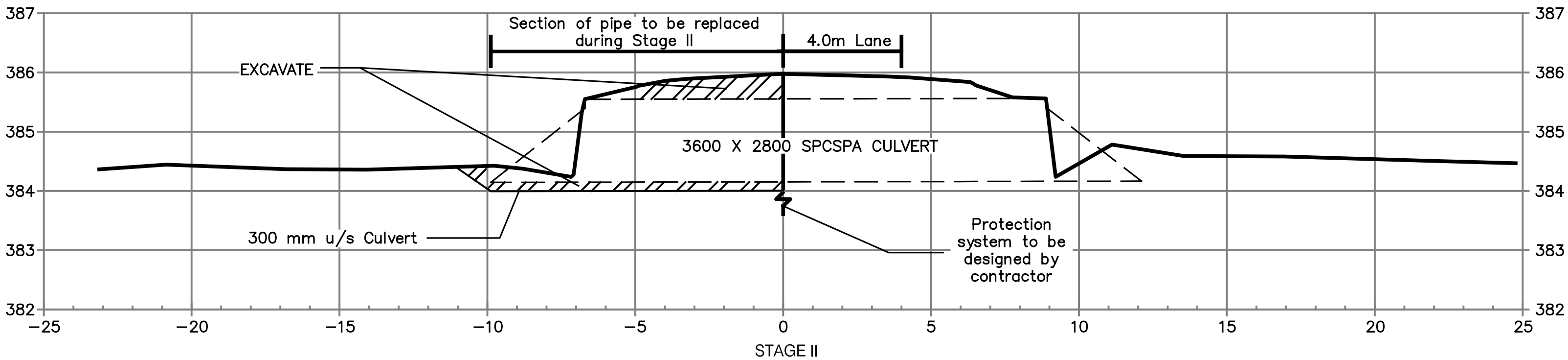
Dimensions are in meters and/or millimeters unless otherwise shown. Stations are in kilometers + meters.

HWY 560 - Township of Vrooman - Little Mollie Creek Culvert at Station 10+000, Conceptual Shoring Locations	FIGURE SK-4

REF.: 13073-F8

LVM | MERLEX





2.5m 0 5m

METRIC

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

HWY 560 - Township of Vrooman - Little Mollie Creek Culvert at Station 10+000, Conceptual Shoring - Typical Sections

FIGURE SK-5