



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

**Highway 144 Rehabilitation
Culvert Replacement
Elboga Lake Culvert
Station 15+629 – Twp. Of Muldrew
GWP 5468-09-00**

**Highway 144
From 0.3 km North of Halfway Lake Channel Culvert,
Northerly 19.4 km
MTO Sudbury Area**

FINAL FOUNDATION INVESTIGATION AND DESIGN REPORT

Date: March 17, 2014
Ref. N^o: 12/09/12182

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LVM | MERLEX



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Final Foundation Investigation and Design Report

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1 INTRODUCTION

LVM | Merlex Ltd. has been retained by AECOM Canada Ltd., on behalf of the Ministry of Transportation of Ontario (MTO), to carry out a foundation investigation at an existing Structural Plate Corrugated Steel Pipe (SPCSP) centreline culvert site. The site is located at Elboga Lake on Highway 144, some 38.2 km North of the Town of Cartier, in the Township of Muldrew.

The foundation investigation location was specified by the MTO in the Terms of Reference for additional work under Agreement No. 5011-E-0012. The terms of reference for the scope of work are outlined in LVM | Merlex Ltd.'s Proposal 12/09/12182-144, dated May, 2013 and submitted under Change Order No. 3. The purpose of this investigation was to determine the subsurface conditions in the area of the culvert and proposed detour. LVM | Merlex Ltd. 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 Elboga Lake Structural Plate Corrugated Steel Pipe (SPCSP) culvert is located on Highway 144 at Station 15+629, Township of Muldrew. The culvert is located in a bedrock controlled valley, with elevated bedrock outcroppings to the north and south of the culvert. The valley connects Elboga Lake to the east (right) with Shamberson Lake to the west (left). The existing highway embankment currently supports two undivided lanes, running in a north-south direction. The existing highway, at the culvert location, is constructed on a rock fill embankment some 8.1 m in height, with centerline elevation of approximately 431.2 m at the culvert location. The existing embankment slopes are at an average angle of approximately 1.4H:1V. The culvert at this location is reported as a 2.1 m diameter Structural Plate Corrugated Steel Pipe (SPCSP) culvert, some 47.9 m in length. Flow through the culvert is from west to east (left to right) (see Photo Essay, Appendix 4).

Infrastructure at the culvert location consists of overhead wires on the right (east) 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 144 is generally rolling. Significant layers of earth overlay the bedrock. There are numerous bedrock outcroppings throughout the site area. Organic terrain was also observed. Within the project area native overburden consists primarily of a sand and gravel containing various amounts of silt, cobbles, and boulders.

Bedrock in the area, as indicated on OGS Map 2506, is of the Early Precambrian Era consisting of Felsic Igneous and Metamorphic Rocks including; granitic rocks, syenite, pegmatite, and unsubdivided migmatite. At the location of this culvert foundation investigation, the bedrock generally comprises of granitic rock, based on the diamond core drilling carried out at the site.

3 INVESTIGATION PROCEDURES

The field work for this investigation was carried out during the period of August 8th to September 25th, 2013 during which time ten (10) sampled boreholes were advanced. Four (4) boreholes were advanced through the embankment, two (2) boreholes were advanced to the right of the embankment, in the area of the culvert outlet, and four (4) boreholes were advanced along the detour to the left (west) of the embankment with one of these boreholes advanced at the inlet end of the culvert.

The field investigation was carried out using both a Bombardier mounted CME 45 drill rig and a truck mounted CME 75 drilling rig equipped with hollow stem augers, standard augers, NQ size coring equipment and routine geotechnical sampling equipment. Soil samples were obtained from the boreholes 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. At some of the sample depths blow counts of 25 or greater blows for 0 mm penetration were encountered. This sample spoon refusal was shown on the Record of Borehole Logs as “25/0mm” and was due to a cobble or boulder size obstruction at the sample depth. To penetrate the obstructions NQ coring equipment was employed to advance the hole. This diamond core equipment was advanced at a constant feed rate. Because of the numerous voids in the rock fill no wash water was observed returning to the collar of the borehole. All boreholes were advanced to the predetermined depth or 3 metres into bedrock. All samples obtained during this investigation were stored in labeled airtight containers for transport to our North Bay laboratory for visual examination and selected 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 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 surface of the hole, where necessary, was backfilled with an asphalt cold patch to seal the existing asphalt surface.

The field work for this investigation was completed 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, particle size analysis, Atterberg Limits determination, as well as specific

gravity testing. The results of the laboratory testing are presented on the individual Record of Borehole Sheets (Appendix 2), with a summary of results presented on the laboratory sheets in Appendix 3 (Figures Nos. L-1 to L-4).

The location of the individual boreholes were determined in the field using highway centreline chainage (established by others) and offset relative to highway centreline. 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.

4 SUBSURFACE CONDITIONS

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

4.1 ELBOGA LAKE CULVERT

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 7 advanced at the culvert ends (inlet (left/west) and outlet (right/east), respectively), Borehole Nos. 2, 3, 4, and 5 advanced through the embankment, Borehole No. 6 advanced to the right (east) of the embankment, and Borehole Nos. 8, 9, and 10 advanced along the proposed detour alignment to the left (west) of the embankment. At the time of the subsurface investigation, the ground surface elevations at Boreholes Nos. 1 to 10 were recorded at Elevations 424.1, 431.4, 432.7, 429.7, 430.9, 424.9, 424.0, 424.6, 423.4, and 431.6 m, respectively.

4.1.1 Pavement Structure

Borehole Nos. 2 to 5, inclusive, were advanced from the highway shoulder where a layer of asphalt some 100 to 200 mm thick was penetrated. At Borehole No. 2, the asphalt layer was underlain by a layer of crushed gravel approximately 100 mm thick.

4.1.2 Granular Fill

Underlying the pavement structure, at Borehole Nos. 2 to 5 inclusive, a layer of granular fill consisting of brown gravelly sand to gravel and sand trace silt was penetrated. The natural moisture content measured on samples of this deposit was in the order of 7 to 14%. Gradation analyses were carried out on two (2) samples of this deposit, the results of which indicated 39

to 59% gravel size particles, 37 to 53% sand size particles, and 4 to 8% silt and clay size particles (Figure No. L-1, Appendix 3). Based on SPT 'N' values of 26 to 86 blows per 300 mm penetration, the compactness of this deposit was described as compact to very dense. This deposit was encountered to depths of 1.2, 2.0, 0.6, and 0.8 m below grade at Borehole Nos. 2 to 5, respectively (elevations 430.2, 430.7, 429.1, and 430.1 m, respectively).

4.1.3 **Rock Fill**

Underlying the granular fill at Borehole Nos. 2, 4, and 5, a layer of rock fill was penetrated. NQ size coring equipment was used to penetrate the rock fill. The rock fill contained numerous voids, based on the lack of wash water return. Sample recovery was very poor (0 to 10%) in the rock fill. A pocket of gravelly sand was encountered, partially filling a rock fill void, between 3.0 and 3.5 m depth at Borehole No. 2, indicating the rock fill was mixed with sands, gravel, and rock spall. The rock fill was encountered to depths of 5.8, 1.4, and 7.9 m below grade at Borehole Nos. 2, 4, and 5, respectively (elevations 425.6, 428.3, and 423.0 m, respectively).

4.1.4 **Fill**

Underlying the rock fill at Borehole No. 2, a layer of fill described as brown sand and gravel trace silt (likely embedment fill) was penetrated. The natural moisture content measured on samples of this deposit was in the order of 9%. A gradation analysis was carried out on one (1) sample of this deposit, the results of which indicated 43% gravel size particles, 50% sand size particles, and 7% silt and clay size particles (Figure No. L-2, Appendix 3). Based on SPT 'N' values of 3 to 32 blows per 300 mm penetration, the compactness of this deposit was described as very loose to dense, generally compact. This deposit was encountered to a depth of 7.6 m below grade (elevations 423.8 m).

4.1.5 **Surficial Organics**

At surface, at Borehole Nos. 9 and 10, a layer of surficial organics some 50 to 300 mm thick was penetrated.

4.1.6 **Sands**

Underlying the granular fill at Borehole No. 3, underlying the rock fill at Borehole Nos. 4 and 5, underlying the fill at Borehole No. 2, at surface at Borehole Nos. 1, 6, 7, and 8, and underlying the surficial organics at Borehole Nos. 9 and 10, a deposit of sands with varying silt and gravel content was penetrated. Numerous cobbles and boulders were encountered in this deposit, except at Borehole No. 9. Due to the concentration of cobbles and boulders, NQ size casing and coring equipment was required to advance the borehole through this stratum. A 600 mm length of boulder core was retrieved while coring through this deposit. It should be noted that due to the high concentration of cobbles and boulders, and coarse gravel content in this deposit, sample return in the 37.5 mm I.D. split spoon sampler was generally poor. The natural moisture content measured on samples of this deposit was in the order of 6 to 9%. Organics were encountered near surface in this deposit at several boreholes, resulting in elevated moisture contents. Gradation analyses were carried out on ten (10) samples of this deposit, the

results of which indicated 8 to 42% gravel size particles, 31 to 79% sand size particles, and 9 to 49% silt and clay size particles (Figure No. L-3, Appendix 3). Based on SPT 'N' values of 11 to 102 blows per 300 mm penetration, the compactness of this deposit was described as compact to very dense. The high concentrations of cobbles and boulders impacted the SPT values however generally the compactness of the sand stratum can be considered as dense. Sampling was terminated in this deposit at a depth of 8.1 m below grade at Borehole No. 7 (elevation 415.9 m). This deposit was encountered to depths of 5.8, 9.1, 4.4, 9.1, 8.5, 4.9, 3.1, 2.6, and 2.4 m below grade, where bedrock was encountered, at Borehole Nos. 1 to 6, inclusive, and Borehole Nos. 8, 9, and 10, respectively (elevations 418.3, 422.3, 428.3, 420.6, 422.4, 420.0, 421.5, 420.8, and 429.2 m, respectively).

4.1.7 Bedrock

Underlying the sand stratum at Borehole Nos. 1 to 6, inclusive, and Borehole Nos. 8, 9, and 10, bedrock was encountered. NQ size coring equipment was used to retrieve bedrock cores. The bedrock was described as grey granitic rock. Based on Rock Quality Designation (RQD) values of 13 to 91%, the quality of rock was described as very poor to excellent, generally fair quality. Sampling was terminated in the bedrock at depths of 9.1, 12.2, 7.4, 12.2, 11.3, 7.9, 5.9, 5.9, and 5.4 m below grade at Borehole Nos. 1 to 6, inclusive, and Borehole Nos. 8, 9, and 10, respectively (Elevations 415.0, 419.2, 425.3, 417.5, 419.6, 417.0, 418.7, 417.5, and 426.2 m, respectively).

4.2 GROUNDWATER DATA

At the time of this investigation, the water level in the culvert was measured at elevation 424.1 and 422.8 m at the inlet and outlet, respectively.

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. A standpipe was installed in Borehole No. 2, to obtain post borehole completion water levels. At the time the field work was undertaken, the water level in the standpipe at Borehole No. 2 had stabilized at Elevation 422.7 m. Water levels are recorded on the individual Record of Borehole Log Sheets (Appendix B).

Along the detour alignment the water level was measured between Elevations 423.1 to 428.9 m. The latter water level, at Elevation of 428.9 m, indicates the water is perched in a granular pocket on the bedrock in the area of Borehole No. 10. Groundwater was not encountered in Borehole Nos. 3, 4, and 5.

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

5 DISCUSSION AND RECOMMENDATIONS

5.1 GENERAL

A foundation investigation was carried for the proposed culvert replacement at the Elboga Lake Culvert.

The existing culvert, located at Station 15+629 in the Township of Muldrew, is a 2.1 m diameter SPCSP culvert some 47.9 m long. The existing culvert invert is at a depth of some 8.1 m (elevation 423.1 m), at centreline. The existing highway embankment currently supports two undivided lanes, running in a north-south direction. Flow through the culvert is from Shamberson Lake, to the west (left), to Elboga Lake to the east (right). Based on data obtained during this foundation investigation, the embankment supporting the existing pavement structure has been constructed using a granular pavement structure overlying rock fill. The native soils, underlying the embankment fills, consisted of generally dense sands with varying silt and gravel content and a high concentration of cobbles and boulders, with bedrock at a shallow, but variable, depth beneath the embankment.

Based on a hydrological review carried out by AECOM, it is understood that the existing culvert is undersized. It is further understood that, presently, the preferred option is to replace the existing SPCSP culvert with a precast concrete rigid open frame culvert measuring 3.6x3.0 m. However, a closed precast concrete rigid frame culvert or an open footing pipe arch culvert are also considered for culvert replacement. It is understood that the final vertical and horizontal alignment of the highway will remain essentially the same.

5.2 SITE CONSIDERATIONS

The founding native typically dense sands with varying silt and gravel content, with a high concentration of cobbles and boulders overlying shallow bedrock, present below the existing embankment are considered excellent 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. As such, site conditions (i.e. embankment composition, native soils, groundwater conditions, etc.) will be the challenging factors for culvert design.

The composition of the rock fill embankment will be one constraining factor for design. Since the existing embankment has been constructed of rock fill, advancing culverts/pipes (permanent culvert or temporary bypass pipe) through the embankment using trenchless technologies would be challenging, and costly. Similarly, open excavation through the rock fill embankment for installation of a bypass culvert, located to either side of the existing culvert, would also be challenging. While using the existing culvert for bypass pumping can be considered, this limits the options for culvert replacement to those with an open bottom.

Another constraining factor for design will be the very dense native material, which contains numerous cobbles and boulders. While the native soils overlying bedrock are considered

excellent for bearing, the typically dense soils may cause difficulties during construction and will require equipment of sufficient capacity to excavate the native soils. If equipment of insufficient capacity is used, disturbances of the subgrade could occur, resulting in potential delays due to additional excavation, etc. Additionally, the potential for disturbance will be greater with deeper excavations, as more cobbles and boulders would be encountered requiring removal. As such, shallow excavations are recommended to minimize the potential for disturbances. Additionally, bedrock was encountered at a shallow depth below the center of the culvert, however, was encountered at greater depths, progressing to either end of the culvert.

The third constraining factor at this site is the groundwater/groundwater control. Generally a cofferdam system is used to allow excavations for culvert replacement. However, due to the dense soils with cobbles/boulders, advancing sheet piling for use as a cofferdam is not practical at this site. While gravity type cofferdams can be used, carrying out excavations to substantial depths below the existing stream base may be problematic with gravity type cofferdams. Diversion of the flow must also be considered during culvert replacement, and, as noted previously, the installation of a bypass culvert has its own challenges.

A discussion of the culvert replacement options is provided in the following Section 5.3.

5.3 CULVERT REPLACEMENT OPTIONS

5.3.1 Open Frame Culvert

As previously noted, the preferred option is to replace the existing SPCSP culvert with a precast concrete rigid frame open culvert measuring 3.6x3.0 m. The open frame culvert would be founded on concrete footings. A structural plate corrugated steel pipe arch culvert on concrete footings has also been considered, and is included in this discussion.

An open frame culvert is the preferred method, as this would allow bypass pumping through the existing culvert during culvert replacement. To carry out construction of an open footing culvert, an open excavation would be required to carry out the replacement. During construction of the strip footings for the culvert, the existing CSP can be left in place and the new footings constructed on either side.

If an **open** footing culvert (i.e. precast concrete rigid frame open or open steel arch on strip footings) is considered, then a factored Geotechnical resistance at ULS of 300 kPa, and a Geotechnical reaction at SLS of 275 kPa, in consideration of 25 mm settlement, would apply for design of strip footing foundations in the native dense soils below the existing culvert. These values take into consideration the limited depth of overburden above the footing (inside footing) and small footing width.

Footings for open culverts must also be protected against the detrimental effects of scour and frost heave. As per the CHBDC the minimum depth of footing for scour protection is not less than 1.5 m, below the original stream bed. The MTO Concrete Culvert Design and Detailing Manual, August 2003, Division 5 RFO Culverts, indicates that the standard footing depth is 1.2

m or equal to the minimum depth of frost protection in Ontario. This implies the depth of footing is to be increased to match the depth of frost penetration at the specific site. At Elboga, the frost penetration below pavement is estimated at 2.2 m and below an unheated structure is some 1.9 m (however is probably substantially less when snow cover and heat loss from the stream flow is factored in).

Excavations to these depths, or greater, will require a sophisticated groundwater control system to maintain the native cohesionless soils, (cobble and boulders in a sand and gravel matrix), in an unwatered stable condition during construction. Adequate penetration of a sheet pile system, for groundwater control, is not feasible due to the high concentration of cobbles and boulders in the underlying native soils and the dense state of compactness. Installation of well points or larger diameter wells would also encounter difficulties, due to the cobbles and boulders, and would probably require, at a minimum, a duplex rotary percussion drilling system to install the filtered well points with filter packs to the proper depth.

Besides foundation depth, scour protection can also be supplied through the placement of R-50 Rip Rap, 500 mm deep to line the stream bed, in the bottom of the open culvert, with an apron of Rip Rap, extending 5 m beyond the inlet and outlets, 5 m either side out the culvert ends and up the bank to 500 mm above the high water level. This protection would allow the footings to be founded at a higher elevation. However, raising the footings will place them within the potential depth of frost penetration and alternative means of frost protection (i.e. non-frost susceptible fill, synthetic insulation, etc.) may have to be considered, or the culvert designed to withstand the possible movements due to differential frost heaving.

The depth of excavation required for scour and frost protection of strip footings, supporting an open culvert, can also be greatly reduced if the footings are supported on piles. If local scour develops the load is carried by the piled foundation. A conventional driven H pile or pipe pile deep foundation is not considered appropriate for this site due to the limited pile length and presence of cobbles and boulders in the underlying soils, which would adversely limit pile penetration. In our opinion a more appropriate type of piled foundation would consist of a micropile type system drilled into the underlying bedrock.

5.3.1.1 ***Micropiles***

A micropile is a small diameter (typically less than 300 mm diameter) pile and consists of either a cased or uncased cement grouted column, with one or more centrally located high tensile strength steel threaded rod(s) or pipes. The capacity of the micropile is dependent upon the bond between the grouted portion of the micropile and the bedrock and/or soil. It should be noted that the design of a micropile foundation is proprietary and there are many different micropile configurations and installation methods that may be used to meet different founding and loading conditions.

At this site, it would be possible to advance a micropile through the overburden and into the bedrock. Since the thickness of overburden varies across this site, the capacity of the micropile

would be based on the grout to bedrock bond length, probably ignoring the contribution of the variable overburden thickness. A duplex rotary percussive drill rig with a down the hole hammer (DTH) would be the preferable method to advance the casing through the cobble and boulder overburden into the bedrock. Once the bedrock is encountered and the casing sealed into bedrock, a smaller diameter bit would be lowered down the casing to allow percussive drilling into the bedrock. Once the required penetration into bedrock is achieved the drill rod is removed and a high strength threaded rod (i.e. Dywidag Threadbar, Williams Anchor, or equivalent) is lowered into the bedrock socket. Tremie methods can then be used to grout the rod into the bedrock from the bottom of the hole up to underside of footing elevation, using neat cement grout, typically with a compressive strength of 30 MPa strength or greater. In this case the geotechnical resistance of the micropile would be limited to the Grout-to-Bedrock bond. A micropile design is presently based on the Federal Highway Administration publication FHWA-SA-97-070, June 2000.

In the central part of the culvert, where the bedrock is high, the strip footing/grade beam could be supported directly on the bedrock, and dowelled in as required. A preliminary design bearing pressure of 1500 kPa can be used for strip footings supported on a sound bedrock surface. As the footing extends towards the inlet and outlet ends, where the bedrock surface dropped off, a system of micropiles could be installed on a longitudinal interval of some 2.5 to 3 m. The micropiles would be drilled through the native soils and a minimum 3 m into bedrock. The centre high tensile strength steel threaded rod(s) of the micropile would be grouted into the bedrock and extended up, through the overburden, and into the footing. The micropiles can be designed to withstand various compression and tension loads, depending upon depth of penetration/bond length in the bedrock. The piles can be designed to accommodate local scour and a water resistant void form can be placed between the underside of the grade beam and subgrade to accommodate the possible heave due to frost penetration.

As noted previously micropiles are proprietary and the specific design is carried out by the contractors engineering personnel. The specialist foundation contractors have different drilling equipment and tooling size however they will be able to satisfy the contract if imposed loads and load location are supplied on the contract drawings. As a base/typical design the factored geotechnical resistance of a micropile constructed with a #14 (43 mm diameter) Dywidag, Grade 75 threaded rod (or equivalent), gravity grouted into a 100 mm diameter by 3.0 m long bedrock socket can be taken as 600 kN. This geotechnical resistance does not account for the upper 0.5 m of rock socket, due to the occasional poor quality rock designation.

The design of the micropile system must be carried out by a Professional Engineer registered in the Province of Ontario. The contractors design must be verified with a minimum of one static axial load test, on a sacrificial pile, in accordance with ASTM test procedure. Considering the pile is socketed into bedrock we suggest that a verification Tension axial load test, as per ASTM D 3689, be specified verses a Compression axial load test.

5.3.1.2 Open Frame Culvert Recommendations

The following table contains the advantages and disadvantages of the different foundation options for open frame culverts.

Table 1: Comparison of foundation alternatives

METHOD	ADVANTAGES	DISADVANTAGES	COSTS
Strip Footings at Depth for Scour and Frost Protection	<ul style="list-style-type: none"> Footings at depth below stream bed are protected against scour and frost action. 	<ul style="list-style-type: none"> Extensive dewatering required during foundation construction in higher permeable soils. Installing dewatering system would be complex and costly due to cobbles and boulders in granular matrix below footing locations. Cost of concrete foundation wall and footings below culvert walls. 	<ul style="list-style-type: none"> Well system to allow excavation to frost/scour depth – Additional cost \$165,000.00
Micropile Supported Foundations	<ul style="list-style-type: none"> Piles can be designed to accommodate local scour and frost action. Provides high bearing and uplift capacities. Can be installed from a shallow depth, greatly simplifying costly dewatering requirements. Reduces concrete quantities over long haul distances. Greatly reduced below groundwater excavation quantities. 	<ul style="list-style-type: none"> Cost of micropile system with grade beam 	<ul style="list-style-type: none"> 27 micropiles to support grade beam to bedrock - \$100,000.00

Based on the above review and in consideration of the potential difficulties with dewatering during installation of conventional strip footings, resulting in potential delays and claims, a micropile foundation is recommended for an open culvert installation.

5.3.2 Closed Culvert

A closed culvert, such as a precast concrete rigid frame box culvert, was also considered for culvert replacement. The installation of a closed box culvert would also require open cut excavation with detour construction. To install a closed box culvert, the existing culvert would likely have to be removed during excavation, unless the new culvert can be aligned to the one

side of the existing culvert. As such, bypass pumping, through a temporary culvert would be required to allow replacement. Considering this embankment has been constructed of rock fill, the recommended method of installation of a temporary bypass pipe would be to install the pipe through the existing embankment using an open cut excavation. The bypass culvert should be installed outside the area of excavation for the culvert replacement, at a depth of some 1 m below the existing road surface. A bypass pipe would also be required through the proposed detour. In consideration of the anticipated flow to be diverted, several large capacity pumps would be required to divert the flow, along with a backup system. As such, a closed box culvert has the disadvantage of requiring a more complex and costly bypass system.

Based on the characteristics of the native typically dense sand subgrade present below the existing culvert embedment, bedrock elevation, the response of the existing embankment, and a founding elevation similar to that of the existing culvert, a factored Geotechnical resistance at ULS of 700 kPa can be used for a **closed** culvert (i.e. precast concrete rigid frame box (RFB) culvert). In consideration of the width of the culvert and depth of overburden a geotechnical reaction at SLS of 425 kPa can be used for design of a closed culvert, in consideration of 25 mm settlement, where deeper overburden is present.

The results of Boreholes No. 2 and No. 5 indicate that the bedrock is at Elevation 422.3 m and 422.4 m, respectively. The invert of the existing culvert, at highway centreline, is at elevation 423.1 m, which is some 700 to 800 mm above the bedrock, in the central part of the culvert. At Borehole No. 1 (culvert inlet) the bedrock was proven at Elevation 417.8 m. Bedrock was not encountered at Borehole No. 7 (culvert outlet) within the depth of the boring, and as such, the bedrock at this location is below Elevation 415.9 m (elevation where borehole was terminated). The native soils underlying the culvert consist of typically dense sands and gravels with a high concentration of cobbles and boulders.

The centre portion of the new culvert will be at or very close to the surface of the unyielding bedrock, whereas the inlet and outlet will be underlain by overburden, greater than some 5.3 m thick. Generally uniform subgrade conditions, either all yielding (overburden) or all unyielding (bedrock), are desirable to minimize the potential for differential settlement. At this location, a yielding subgrade could be achieved by removing bedrock to a minimum depth of 600 mm below the final culvert founding elevation and building the subgrade up to the underside of the culvert base with a pad of granular fill. This approach could be considered for a rigid frame precast box structure. The cushion of granular fill over unyielding bedrock would limit differential settlement to the generally acceptable 25 mm settlement along the length of the culvert provided the native sand subgrade is not excessively disturbed during construction.

Engineered fill should consist of a 600 mm thick layer of Granular A or Granular B Type II compacted to a uniform density of 100 % Standard Proctor Dry Density (SPDD). The layer of engineered fill should extend beyond the edges of the footing, a horizontal distance of 300 mm and then downward on a 45 degree angle to the bedrock surface. The engineered fill should be placed in lifts of 200 mm thickness and compacted to 100% SPDD. Provided the engineered fill

is properly compacted and the native dense cohesionless deposits are not excessively disturbed during construction, settlements will be within the limit noted above for the geotechnical reaction at SLS.

5.3.3 Trenchless/Tunnelling Techniques

The boreholes through the embankment indicate that random sized rock fill, containing voids, has been used to construct the existing embankment at Elboga Lake. The embankment is over 8 m in height. As such a trenchless approach to culvert replacement would eliminate the need for a detour and associated traffic delays. Several trenchless technologies are available, as outlined in the following Table 1. However, the rock fill composition of the embankment will limit the type of trenchless method that can be used at this site. The following table contains the advantages and disadvantages of the different trenchless techniques.

Table 2: Comparison of trenchless techniques

METHOD	ADVANTAGES	DISADVANTAGES
Horizontal Direction Drilling	<ul style="list-style-type: none"> Can be used in most ground condition Generally does not require staging pits therefore minimal ground water control required Alignment can be adjusted to avoid obstructions 	<ul style="list-style-type: none"> Site grades may require longer bore or staging pits Larger drilling equipment may be required Requires drilling fluid to maintain the bore, which could result in heave Size of pipe limited to 140 to 1200 mm
Symmetrix Drilling	<ul style="list-style-type: none"> Can be advanced through bedrock, and most overburden types has been used to advance casings through many rock fill in Scandinavian countries 	<ul style="list-style-type: none"> Size limited to 140 to 1200 mm
Jack and Bore	<ul style="list-style-type: none"> Good contractor availability Good for shorter tunnel length (<100 m) Good gradient control 	<ul style="list-style-type: none"> Requires entrance and exit pits/shafts Groundwater control is required for the bore and shafts Elevated potential for ground subsidence Boring diameter 1 m plus required to allow removal occasional cobbles/boulders Diameter range generally 200 to 1500 mm Not applicable in rock fills or high concentrations of large obstructions

METHOD	ADVANTAGES	DISADVANTAGES
Micro-Tunneling/Pipe Jacking	<ul style="list-style-type: none"> Shield face can accommodate high groundwater conditions Can accommodate cobble/boulders with appropriate shield Can advance boring from 3.2 m diameter maintenance hole in urban areas Alignment can be altered during bore. 	<ul style="list-style-type: none"> Requires staging pit/shaft possible with groundwater control Requires thrust block of sufficient mass to jack pipe. Generally only economical at diameters of 1 m or greater. Has not been proven to penetrate through rock fill.
Pipe Ramming	<ul style="list-style-type: none"> Minimal groundwater control required along the installation route (unless required to remove obstruction/old pipe) Can penetrate soils containing cobbles/boulders if obstruction less than casing diameter. Has been used successfully in the USA to penetrate a railway rock fill embankment 	<ul style="list-style-type: none"> Installation problems can occur in soft soils with cobble/boulders Requires staging pits/shafts Groundwater control is required for staging pits Possible ground subsidence in very loose soils Size of pipe is generally limited to less than 2.0 m diameter, although Contractors are developing methods to increase up to 3.0 m plus

To date there are no known projects in Ontario where trenchless methods have been used to install casing through rock fills. However, the Symmetrix System, as described above, is a proven technology and has been used for some 8 years in Scandinavian countries for advancing horizontal holes, up to 1200 mm diameter, through rock fill.

Pipe Ramming could also be considered, for advancing a heavily reinforced casing through rock fill. However, to Pipe Ram the casing must be large enough to allow hand mining operation to address large pieces of rock fill that could not be swallowed into the advancing casing. Generally a 1.2 m diameter is required to have sufficient room to hand mine rock fill. The following is a brief discussion of the two methodologies.

5.3.3.1 *Symmetrix*

The Symmetrix system has been used in Ontario for advancing vertical holes, some 1 m diameter, through solid rock and rock fills. However, to date, it has not been used in a horizontal direction. An advantage of the Symmetrix system is that it can accommodate uneven bit pressures, which frequently develop when advancing through rock fills containing voids. The Symmetrix system has been proven, outside Canada, for casings up to 1200 mm diameter in granite type rock fills and therefore warrants consideration for testing in Ontario.

5.3.3.2 *Pipe Ramming*

The other trenchless technology that has been considered for advancing a casing through a rock fill embankment would be to employ pipe ramming techniques. Based on discussions with

a North American supplier of pipe ramming equipment, it is understood that recently 1.2 m diameter casing had been rammed through a rock fill railway embankment in Colorado. However this method has not been undertaken in Ontario. If undertaken, the size of the casing would have to be large enough to allow a person to enter to hand mine pieces of rock fill that are larger than the casing. As such, pipe ramming through rock fill could be considered for a larger diameter casing (i.e. greater than 1.2 m diameter). The high compressive strength of granitic rock would also require a robust, heavily reinforced casing shoe. Based on the above, pipe ramming, on a trial basis, could be considered.

5.3.3.3 *Trenchless Technique Recommendations*

As noted above, the existing CSP culvert is 2.1 m in diameter. At the commencement of this project LVM | Merlex Ltd. was requested to review trenchless approaches to culvert replacement consisting of lining the existing culvert and advancing a secondary culvert, some 1 m or 1.2 m diameter, parallel to the existing, to accommodate the reduced capacity associated with lining. However, upon review of the geotechnical and hydrological conditions at this site, the MTO has determined that trenchless techniques are not feasible for this project and will employ open cut methods, with detour, to replace the existing culvert. It is understood that several 1 m diameter culverts or a single larger diameter culvert would be required to meet the hydrological requirements. Installing a large diameter culvert or several small culverts through the rock is not considered an effective method for culvert replacement.

5.4 **CULVERT DESIGN, BEDDING, AND EMBEDMENT**

The embankment generally consists of rock fills, with some of the voids partially filled with rock spall, sands, and gravels. The results of this investigation indicate that, below the culvert invert, the native soils consist of compact to very dense sand with varying silt and gravel content, with a high concentration of cobbles and boulders. A review of the condition of the pavement surface, at the culvert locations, revealed minor transverse asphalt cracking, however, in general, the embankment appears to have performed well. The existing embankment has preloaded the limited depth of soils at the culvert location and since there will be no change in the height of the embankment, therefore no increases in embankment load, no appreciable settlement of the embankment is anticipated. As such, installing the culvert on a camber will not be required at this site.

5.4.1.1 *Precast Rigid Frame Box Culvert*

As noted above, a precast rigid frame box culvert was considered for culvert replacement at this site. Bedding for a rigid frame box culvert shall consist of Granular A with a thickness of 300 mm. The bedding under the middle third of the box unit base should be loosely placed and uncompacted. The upper 75 mm portion of the Granular A bedding should be uncompacted throughout the length/width of the box and incorporated as the top levelling course in conformance with OPSS 422. Alternatively, specifically if construction is carried out under wet

conditions, a 19 mm clear stone bedding and levelling course can be used in place of the Granular A, which would aid in dewatering applications.

During cover placement and backfilling the embankment fill should be placed in a balanced manner on the outer sides of the box culvert. The elevation difference of the backfill on either side of the box culvert must be limited to a maximum of 300 mm. Backfilling and construction of precast concrete box culverts shall be in accordance with OPSS 422. Cover material for concrete box culverts can consist of Granular A, placed to the dimensions as shown on MTOD-803.021.

Cut-off walls, 1.2 m deep, must be added to the ends of the rigid frame box culvert in accordance with the MTO Concrete Culvert Design Manual. An apron of rip-rap protection would be required at the culvert ends, for a distance of 5 m either side of the culvert and extend out 5 m. Rip Rap shall meet the requirements of OPSS 1004 for R-50 Rip-Rap. Clay seals are generally used in cohesionless soils where significant head differences exist between the inlet and outlet of a culvert to prevent flow through the embankment and around the culvert. Clay seals are not considered necessary considering the anticipated water levels and flow at this culvert location.

5.5 CULVERT REPLACEMENT

The embankment at this location is some 8.1 m in height above the culvert invert at the centerline. As such, culvert replacement using open cut excavation and staged construction is not considered feasible at this location without the construction of a full detour.

Two different methods for the culvert replacement are discussed in the following. These methods include construction of a temporary detour to allow open cut excavation, and an alternate of employing trenchless technologies for culvert replacement.

5.5.1 Detour

It is understood that a detour is being considered for use during replacement of the Elboga Lake Culvert. It is further understood that the detour will be located to the west of the existing embankment (see Drawing No. 2, Appendix 3). It is anticipated that the detour will be temporary and some 6.5 m in height and, as such, it will not be necessary to strip the topsoil/organics, however the tree and shrub cover should be close cut cleared, in accordance with OPSS 201. Based on the results of this investigation, the existing soils (sands and gravel with cobbles and boulders) are considered acceptable to support the proposed detour embankment, which could consist of up to some 6.5 m of rock fill, depending upon design speed. Embankment construction must be in accordance with OPSS 206.

To carry out the culvert replacement it is proposed to cut back the west side of the existing embankment, at the culvert location, from 1 m west down at a 1.25H:1V slope, leaving a single lane some 5 m in width. Traffic will be controlled by temporary traffic lights. This will allow replacement of the culvert with a temporary extension on the west end. Following which, a

single lane temporary detour will be constructed with rock fill over the new and extended section of culvert. The remainder of the existing embankment will then be removed to allow replacement of the east section of culvert. After completion of the culvert replacement the embankment will be reconstructed to the current alignment and the detour will be removed.

5.5.2 Excavation, Dewatering, and Embankment Reconstruction

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 native materials will have to be cut back to an angle of 2H:1V, possibly shallower, dependent upon the Contractors' chosen method of controlling the groundwater.

The new embankment rock fill slopes may be reconstructed to match the existing rock fill embankment slopes, measured at 1.4H:1.0V but shall not be steeper than 1.25H:1V as per OPSD – 201.010.

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

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

The water level in the creek was recorded between elevations 424.1 to 422.8 m at the time of this investigation and the water level in Borehole No. 2 (adjacent to the stream) had stabilized at elevation 422.7 m at the time of this investigation. All excavations extending below the groundwater table, present at the time of construction, will have to be maintained in a dewatered condition. During construction, installation of filtered sumps and pumping from the base of the excavation will, at a minimum, be required to maintain the excavation in a dewatered condition during subgrade preparation and culvert installation. The effectiveness of this method of groundwater control would be limited to conditions where the prevailing groundwater table is less than some 1 m above the final excavation depth. If the excavation must penetrate to a greater depth below the prevailing groundwater table a more effective groundwater control method, such as a vacuum well point system, should be considered by the contractor to maintain a stable excavation base.

A sand bag cofferdam or aquadam could be considered for controlling stream flow, depending upon anticipated flow at time of construction. Considering the dense native soils contained cobbles and boulders, a sheet pile coffer dam is not considered feasible for this site, since

adequate penetration of the sheets would be restricted by the presence of boulders. The presence of cobbles and boulders would also adversely impact the installation of a well point system. As noted previously, it is probable that if filtered well points are considered they may have to be installed using a duplex rotary percussive drill to penetrate the native course soils to sufficient depth.

By-pass pumping through a temporary culvert could be carried out to divert the stream flow at the time of construction. However this would require a large pumping capacity, with back-ups, and would be complicated due to the construction of the detour. As such, the approach of using the existing culvert as a bypass and constructing an open rigid frame culvert overtop the existing 2.1 m SPCSP may be a more appropriate approach.

Ultimately, the method of excavation, dewatering, and stream flow diversion will be the choice of the contractor; however the importance of maintaining the subgrade in a dewatered stable condition during excavation and construction operations cannot be stressed enough.

5.5.2.1 **Bedrock Excavation**

Bedrock was encountered at a shallow depth below the existing culvert invert. Depending on the invert and type of 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 and isolated nature of the site.

5.6 **SLOPE STABILITY**

It is understood that the proposed detour will be constructed of rock fill to the standard stable angle of 1.25H:1.0V. It is further understood that the existing rock fill embankment will be cut back to a similar angle of 1.25H:1.0V during construction.

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 slopes of 1.25H:1V in rock fill for both the temporary embankment and final highway embankment. For the purposes of these analyses, the materials were modeled using the following parameters;

PARAMETER	MATERIAL	
	ROCK FILL	SANDS
Unit Weight (kN/m ³)	18.5	19
Effective Friction Angle (degrees)	43	32

The unit weights and friction angles for the slope calculations are based on general representative values for soil types, obtained through laboratory testing and tactile analysis. The results of the analysis indicated a factor of safety in the order of 1.45 against a shallow

failure through the underlying native sand and gravel subgrade (see Figure Nos. S-1 and S-2, Appendix 5). The factor of safety against a deeper seated “global” failure is greater. As such, the stability of the detour and existing embankment will not be an issue provided it is properly constructed as outlined in Section 5.5.2.

Although slopes at an angle of 1.25H:1V are stable in rock fill, care must be exercised when the existing embankment is cut back, since large pieces of rock fill (size could be up to 1/3 the height of the fill) are permitted in the embankment. Since these larger pieces are generally in the lower portions of the fill embankment, the stability of the excavation face may be compromised if care is not exercised during removal.

5.7 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	ROCK FILL	GRANULAR FILL	NATIVE SANDS
Unit Weight (kN/m ³)	22.8	21.2	18.5	20	19
Angle of Internal Friction	34°	31°	43	30	32
Shear Strength (kPa)	-	-	-	-	-
Coefficient of Active Earth Pressure (Ka)	0.28	0.32	0.19	0.33	0.31
Coefficient of Passive Earth Pressure (Kp)	3.54	3.12	5.29	3.00	3.25
Coefficient of Earth Pressure at Rest (Ko)	0.44	0.48	0.31	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.

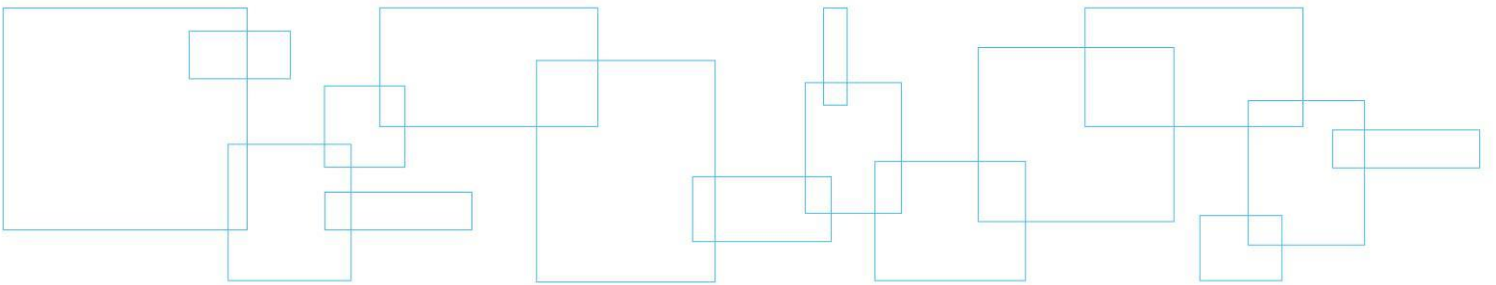
5.8 CONSTRUCTION CONCERNS

It is understood that the existing rock fill embankment will be cut back to a standard angle of 1.25 H: 1.0V during construction. As previously noted, slopes built at this angle are stable in rock fill. However, care must be exercised when the existing embankment is cut back, since large pieces of rock fill (up to 1/3 the fill height) are permitted in the embankment. Since these larger rock fill pieces are generally in the lower portions of the fill embankment, the stability of the excavation face may be compromised if care is not exercised during removal.

Appendix 1 Key Plan

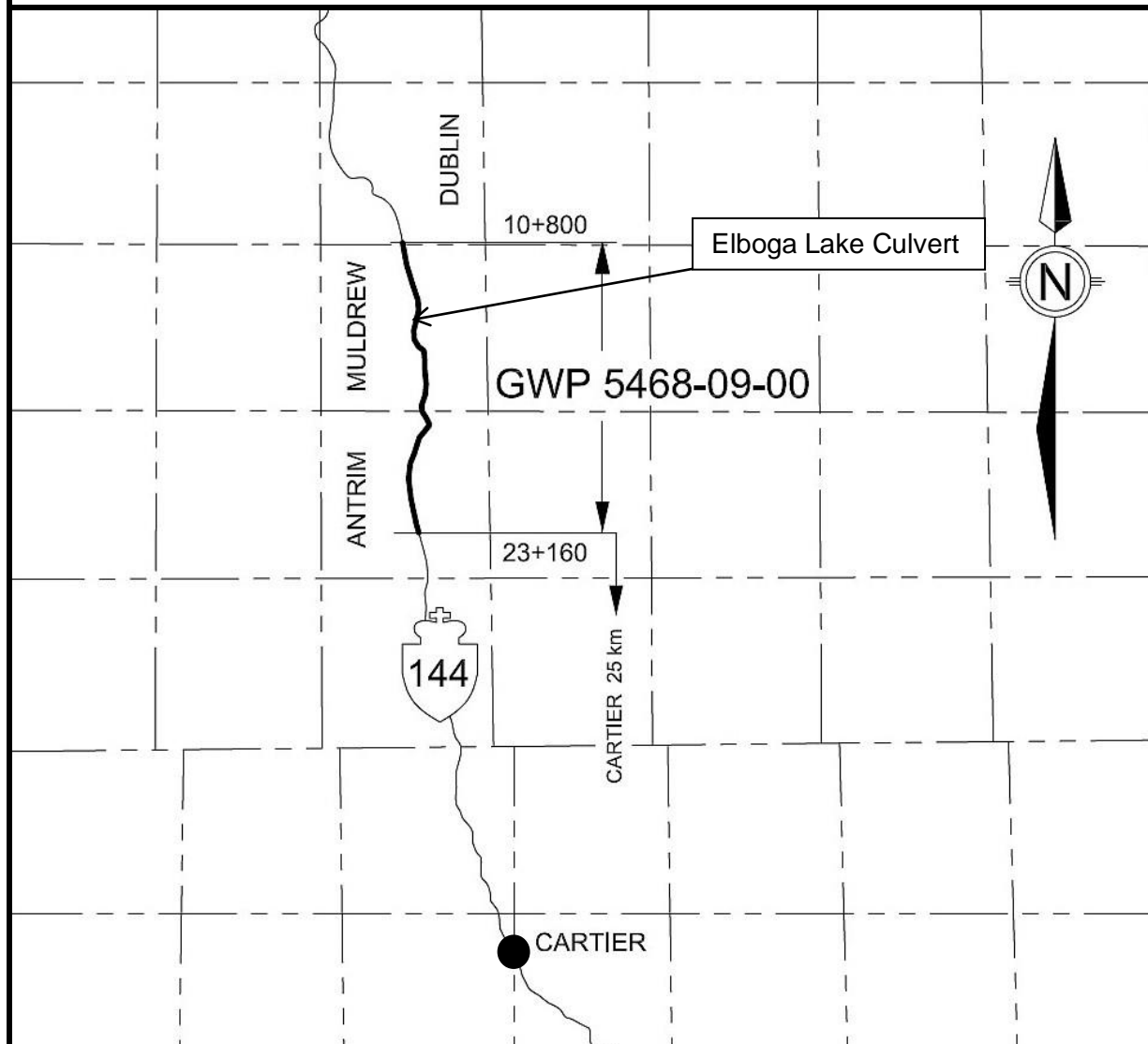
Drawing No. 1

Key Plan



KEY PLAN

NOT TO SCALE



**FINAL
FOUNDATION INVESTIGATION
AND DESIGN REPORT
GWP 5468-09-00**

Highway 144 – Elboga Lake Culvert
From 0.3 km North of Halfway Lake
Channel Culvert, Northerly 19.4 km

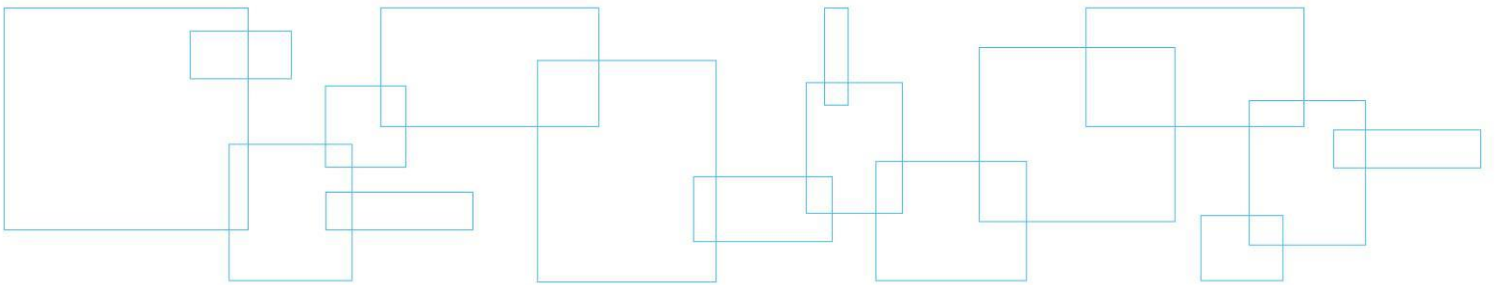
LVM Reference: 12/09/12182

March 2014

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Appendix 2 Subsurface Data

Enclosure No. 1	List of Abbreviations and Symbols
Enclosure Nos. 2 to 11	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

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.

METRIC

RECORD OF BOREHOLE NO. 01



REFERENCE	12/09/12182	DATUM	Geodetic	LOCATION	N 5209833.4 E 256233.4 - Muldrew Township Station 15+644	ORIGINATED BY	JL
PROJECT	GWP 5468-09-00, Elboga Lake Culvert			BOREHOLE TYPE	Track Mounted CME 45B - NW Casing, NQ Core	COMPILED BY	AT
CLIENT	AECOM Inc.			DATE (Started)	2013 September 4	TIME	
				DATE (Completed)	2013 September 4	(Completed)	
						CHECKED BY	MAM

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa								WATER CONTENT (%)
								○ UNCONFINED	+ FIELD VANE	● QUICK TRIAXIAL						
424.1	Ground Surface							20 40 60 80 100								
0.0	SAND - Cobbles/boulders in sand and gravel matrix trace organics at surface		1	SS	27/150 mm										8 79 (13)	
			2	SS	86/275 mm										42 47 (11)	
			3	SS	70/150 mm 30/25 mm										40 51 (9)	
			4	SS	80/125 mm											
			5	RC	Rec= 10%											
			6	RC	Rec= 10%											
			7	RC	Rec= 10%											
418.3	BEDROCK - grey granitic rock		8	RC	Rec= 100% RQD= 83%											
5.8	fair to good quality		9	RC	Rec= 100% RQD= 58%											
			10	RC	Rec= 100% RQD= 65%											
415.0	End of Borehole															
9.1																
COMMENTS							+ ³ , × ³ : Numbers on right refer to Sensitivity Numbers on left refer to values greater than 120 kPa ○ 3% STRAIN AT FAILURE			WATER LEVEL RECORDS						
										Date (dd/mm/yy)/Time		Water Depth (m)		Cave In (m)		
										1) 13/9/4 5:30:00 PM		-0.05		▽ 2.7		
										2) 13/9/9 10:50:00 AM		-0.05		▽ -		
										3) 13/9/11 9:15:00 AM		-0.05		▽ -		
The stratification lines represent approximate boundaries. The transition may be gradual.																

MEL-GEO 12182 - BOREHOLE LOGS.GPJ MEL-GEO.GDT 14/3/17

METRIC

RECORD OF BOREHOLE NO. 02



REFERENCE 12/09/12182 DATUM Geodetic LOCATION N 5209811.2 E 256241.7 - Muldrew Township Station 15+633 ORIGINATED BY JL
 PROJECT GWP 5468-09-00, Elboga Lake Culvert BOREHOLE TYPE Truck Mounted CME 75 - NW Casing, NQ Core COMPILED BY AT
 CLIENT AECOM Inc. DATE (Started) 2013 August 8 TIME 2013 August 8 (Completed) CHECKED BY MAM

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100 WATER CONTENT (%) 20 40 60 PLASTIC LIMIT W _p NATURAL MOISTURE CONTENT W LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA (SI CL)
ELEV DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	"N" VALUES					
431.4	Ground Surface									
0.0	150 mm Asphalt 100 mm Crushed Gravel FILL - brown gravel and sand trace silt (very dense)		1	SS	63					
430.2			2	SS	69					59 37 (4)
1.2	ROCK FILL cobble/boulder size rock fill		3	SS	25/0mm					
			4	SS	25/0mm					
	cobble/boulder size rock fill									
	void and 200 thick layer of gravelly sand encountered between 3.0 to 3.5 m depth		5	SS	11					
			6	SS	25/0mm					
	cobble/boulder size rock fill									
			7	SS	25/0mm					
	cobble/boulder size rock fill									
425.6										
5.8	FILL - brown sand and gravel trace silt (likely embedment granulars) (very loose/dense)		8	SS	3					
			9	SS	32					43 50 (7)
423.8	100 mm piece of asphalt SAND - cobble/boulder size rock in sand and gravel matrix (very dense)		10	RC	REC= 77%					
7.6										
			11	SS	50/100 mm					
422.3	BEDROCK - grey granitic rock poor to good quality		12	RC	REC= 92% ROD= 68%					
9.1										
Continued Next Page										
COMMENTS 1) No water return to surface during coring operations 2) Core advanced at constant rate								+ 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) 13/8/8 12:30:00 PM 8.8 - 2) 13/8/8 4:10:00 PM 8.7 - 3) 13/8/9 8:30:00 AM 8.7 -		

MEL-GEO 12182 - BOREHOLE LOGS.GPJ MEL-GEO.GDT 14/3/17



METRIC

RECORD OF BOREHOLE NO. 02



REFERENCE 12/09/12182 DATUM Geodetic LOCATION N 5209811.2 E 256241.7 - Muldrew Township Station 15+633 ORIGINATED BY JL
 PROJECT GWP 5468-09-00, Elboga Lake Culvert BOREHOLE TYPE Truck Mounted CME 75 - NW Casing, NQ Core COMPILED BY AT
 CLIENT AECOM Inc. DATE (Started) 2013 August 8 TIME
 DATE (Completed) 2013 August 8 (Completed) CHECKED BY MAM

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE			"N" VALUES	20	40	60	80	100	w_p	w		
	Continued from Previous Page															
	BEDROCK - grey granitic rock poor to good quality		13	RC	REC= 100% RQD= 88%											
			14	RC	REC= 100% RQD= 45%											
419.2 12.2	End of Borehole															

MEL-GEO 12182 - BOREHOLE LOGS.GPJ MEL-GEO.GDT 14/3/17



METRIC**RECORD OF BOREHOLE NO. 03**

REFERENCE 12/09/12182 DATUM Geodetic LOCATION N 5209830.8 E 256258.9 - Muldrew Township Station 15+659 ORIGINATED BY JL
 PROJECT GWP 5468-09-00, Elboga Lake Culvert BOREHOLE TYPE Truck Mounted CME 75 - NW Casing, NQ Core COMPILED BY AT
 CLIENT AECOM Inc. DATE (Started) 2013 August 8 TIME
 DATE (Completed) 2013 August 8 (Completed) CHECKED BY MAM

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							
							20	40	60	80	100	20	40	60	
432.7 0.0	Ground Surface 100 mm Asphalt														
	FILL - brown sand and gravel trace silt (compact/very dense)		1	SS	86										
			2	SS	26										
			3	SS	36										
430.7 2.0	SAND - cobble/boulder in sand and gravel matrix														
			4	SS	32										
			5	SS	25/0mm										
			6	SS	25/0mm										
428.3 4.4	BEDROCK - grey granitic rock fair to good quality														
			7	RC	REC= 100% ROD= 87%										
			8	RC	REC= 100% ROD= 63%										
425.3 7.4	And of Borehole														

COMMENTS		+ 3, × 3 : Numbers on right refer to Sensitivity Numbers on left refer to values greater than 120 kPa		WATER LEVEL RECORDS	
1) No water return to surface during coring operations 2) Core advanced at constant rate		○ 3% STRAIN AT FAILURE		Date (dd/mm/yy)/Time	Water Depth (m)
				1) 13/8/8 6:30:00 PM	1.9
				2)	-
				3)	-

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

MEL-GEO 12182 - BOREHOLE LOGS.GPJ MEL-GEO.GDT 14/3/17



METRIC

RECORD OF BOREHOLE NO. 04



REFERENCE	12/09/12182	DATUM	Geodetic	LOCATION	N 5209780.9 E 256223.6 - Muldrew Township Station 15+598	ORIGINATED BY	JL
PROJECT	GWP 5468-09-00, Elboga Lake Culvert			BOREHOLE TYPE	Truck Mounted CME 75 - NW Casing, NQ Core	COMPILED BY	AT
CLIENT	AECOM Inc.	DATE (Started)	2013 August 9	TIME		CHECKED BY	MAM
		DATE (Completed)	2013 August 9	(Completed)			

[illegible]

MEL-GEO 12182 - BOREHOLE LOGS.GPJ MEL-GEO.GDT 14/3/17

METRIC**RECORD OF BOREHOLE NO. 04**

REFERENCE 12/09/12182 DATUM Geodetic LOCATION N 5209780.9 E 256223.6 - Muldrew Township Station 15+598 ORIGINATED BY JL
 PROJECT GWP 5468-09-00, Elboga Lake Culvert BOREHOLE TYPE Truck Mounted CME 75 - NW Casing, NQ Core COMPILED BY AT
 CLIENT AECOM Inc. DATE (Started) 2013 August 9 TIME
 DATE (Completed) 2013 August 9 (Completed) CHECKED BY MAM

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100	W _p	W	W _L		
	Continued from Previous Page																
	BEDROCK - grey granitic rock poor to good quality		10	RC	REC=100% ROD=86%												
			11	RC	REC=100% ROD=44%												
417.5 12.2	End of Borehole																

MEL-GEO 12182 - BOREHOLE LOGS.GPJ MEL-GEO.GDT 14/3/17



METRIC

RECORD OF BOREHOLE NO. 05



REFERENCE 12/09/12182 DATUM Geodetic LOCATION N 5209840.1 E 256202.2 - Muldrew Township Station 15+625 ORIGINATED BY JL
 PROJECT GWP 5468-09-00, Elboga Lake Culvert BOREHOLE TYPE Truck Mounted CME 75 - NW Casing, NQ Core COMPILED BY AT
 CLIENT AECOM Inc. DATE (Started) 2013 August 9 TIME
 DATE (Completed) 2013 August 9 (Completed) CHECKED BY MAM

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100	PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT W _p W W _L WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA (SI CL)
ELEV DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	"N" VALUES						
430.9	Ground Surface										
0.0	200 mm Asphalt		1	SS	27						39 53 (8)
430.1	FILL - brown sand and gravel trace silt (compact)										
0.8	ROCK FILL - rock fill with spall, gravel trace sand trace silt (some voids)		2	SS	26		430				
			3	SS	19		429				
	75 mm long piece of core		4	SS	17		428				
	100 mm piece of core		5	SS	11		427				
	200 mm piece of core										
			6	SS	3		426				
	void in rock fill		7	SS	25/0mm		425				
			8	SS	36		424				
							423				
423.0	SAND - cobble/boulder size rock in sand and gravel matrix		9	SS	50/100 mm						
7.9			10	RC	REC= 100%						
422.4	BEDROCK - grey granitic rock										
8.5	good to excellent quality		11	RC	REC= 100% RQD= 91%		422				
	Continued Next Page						421				

COMMENTS

1) No water return to surface during coring operations 2) Core barrel advanced at constant rate

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) 13/8/9 2:25:00 PM	DRY	3.3
2)	-	-
3)	-	-

MEL-GEO 12182 - BOREHOLE LOGS.GPJ MEL-GEO.GDT 14/3/17

METRIC**RECORD OF BOREHOLE NO. 05**

REFERENCE 12/09/12182 DATUM Geodetic LOCATION N 5209840.1 E 256202.2 - Muldrew Township Station 15+625 ORIGINATED BY JL
 PROJECT GWP 5468-09-00, Elboga Lake Culvert BOREHOLE TYPE Truck Mounted CME 75 - NW Casing, NQ Core COMPILED BY AT
 CLIENT AECOM Inc. DATE (Started) 2013 August 9 TIME
 DATE (Completed) 2013 August 9 (Completed) CHECKED BY MAM

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE			"N" VALUES	20	40	60	80	100	W _p	W		
	Continued from Previous Page															
419.6 11.3	BEDROCK - grey granitic rock good to excellent quality		12	RC	REC=100% ROD=77%	420										

MEL-GEO 12182 - BOREHOLE LOGS.GPJ MEL-GEO.GDT 14/3/17



METRIC

RECORD OF BOREHOLE NO. 06



REFERENCE 12/09/12182 DATUM Geodetic LOCATION N 5209799.7 E 256257.5 - Muldrew Township Station 15+635 ORIGINATED BY JL
 PROJECT GWP 5468-09-00, Elboga Lake Culvert BOREHOLE TYPE Track Mounted CME 45B - NW Casing, NQ Core COMPILED BY AT
 CLIENT AECOM Inc. DATE (Started) 2013 September 24 TIME
 DATE (Completed) 2013 September 25 (Completed) CHECKED BY MAM

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							
							20	40	60	80	100	20	40	60	
424.9	Ground Surface														
0.0	SAND - cobble/boulder in sand and gravel matrix trace organics in upper 1 m (compact)		1	SS	16										
			2	SS	25										
			3	SS	73										
				SS	25/0 mm										
				SS	25/0 mm										
				SS	25/0 mm										
				SS	25/0 mm										
420.0	BEDROCK - grey granitic rock														
4.9	very poor to poor quality		4	RC	Rec=60% RQD=13%										
			5	RC	Rec=100% RQ=48%										
417.0	End of Borehole														
7.9															
COMMENTS							+ 3, × 3 : Numbers on right refer to Sensitivity Numbers on left refer to values greater than 120 kPa			WATER LEVEL RECORDS					
							○ 3% STRAIN AT FAILURE			Date (dd/mm/yy)/Time					
										Water Depth (m)					
										Cave In (m)					
										1) 13/9/25 2:35:00 PM					
										2) -					
										3) -					

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

MEL-GEO 12182 - BOREHOLE LOGS.GPJ MEL-GEO.GDT 14/3/17



METRIC

RECORD OF BOREHOLE NO. 08



REFERENCE 12/09/12182 DATUM Geodetic LOCATION N 5209815.7 E 256223.3 - Muldrew Township Station 15+624 ORIGINATED BY JL
 PROJECT GWP 5468-09-00, Elboga Lake Culvert BOREHOLE TYPE Track Mounted CME 45B - NW Casing, NQ Core COMPILED BY AT
 CLIENT AECOM Inc. DATE (Started) 2013 September 5 TIME
 DATE (Completed) 2013 September 5 (Completed) CHECKED BY MAM

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)											
ELEV DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE			"N" VALUES	20	40	60	80	100	W _p	W			W _L										
424.6 0.0	Ground Surface SAND - cobble/boulder in sand and gravel matrix (compact/dense)		1	SS	12																						
			2	SS	25/0 mm																						
			3	SS	70/150 mm																						
			4	SS	57																						
421.5 3.1	BEDROCK - grey granitic rock good to excellent quality		5	RC	Rec=100% RQD=80%																						
			6	RC	Rec=100% RQD=92%																						
418.7 5.9	End on Borehole																										
COMMENTS						+ 3, x 3 : Numbers on right refer to Sensitivity Numbers on left refer to values greater than 120 kPa ○ 3% STRAIN AT FAILURE					WATER LEVEL RECORDS <table border="1"> <thead> <tr> <th>Date (dd/mm/yy)/Time</th> <th>Water Depth (m)</th> <th>Cave In (m)</th> </tr> </thead> <tbody> <tr> <td>1) 13/9/5 3:15:00 PM</td> <td>0.7</td> <td>1.9</td> </tr> <tr> <td>2) 13/9/9 10:50:00 AM</td> <td>0.9</td> <td>-</td> </tr> <tr> <td>3) 13/9/11 9:15:00 AM</td> <td>0.9</td> <td>-</td> </tr> </tbody> </table>					Date (dd/mm/yy)/Time	Water Depth (m)	Cave In (m)	1) 13/9/5 3:15:00 PM	0.7	1.9	2) 13/9/9 10:50:00 AM	0.9	-	3) 13/9/11 9:15:00 AM	0.9	-
Date (dd/mm/yy)/Time	Water Depth (m)	Cave In (m)																									
1) 13/9/5 3:15:00 PM	0.7	1.9																									
2) 13/9/9 10:50:00 AM	0.9	-																									
3) 13/9/11 9:15:00 AM	0.9	-																									

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

MEL-GEO 12182 - BOREHOLE LOGS.GPJ MEL-GEO.GDT 14/3/17



METRIC

RECORD OF BOREHOLE NO. 09



REFERENCE 12/09/12182 DATUM Geodetic LOCATION N 5209788.1 E 256200.2 - Muldrew Township Station 15+590 ORIGINATED BY JL
 PROJECT GWP 5468-09-00, Elboga Lake Culvert BOREHOLE TYPE Track Mounted CME 45B - NW Casing, NQ Core COMPILED BY AT
 CLIENT AECOM Inc. DATE (Started) 2013 September 5 TIME
 DATE (Completed) 2013 September 6 (Completed) CHECKED BY MAM

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)											
ELEV DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100	W _p	W	W _L													
423.4	Ground Surface																											
0.0	300 mm forest mat (organics)		1	SS	15											41 44 (15)												
	SAND - brown sand varying silt and gravel content (compact/dense)		2	SS	36																							
			3	SS	47											18 35 43 4												
			4	SS	50/125 mm																							
420.8	BEDROCK - grey granitic rock																											
2.6	poor to fair quality		5	RC	Rec=100% RQD=67%																							
			6	RC	Rec=100% RQD=42%																							
			7	RC	Rec=98% RQD=56%																							
417.5	End of Borehole																											
5.9																												
COMMENTS								+ 3, x 3 : Numbers on right refer to Sensitivity Numbers on left refer to values greater than 120 kPa O 3% STRAIN AT FAILURE					WATER LEVEL RECORDS <table border="1"> <thead> <tr> <th>Date (dd/mm/yy)/Time</th> <th>Water Depth (m)</th> <th>Cave In (m)</th> </tr> </thead> <tbody> <tr> <td>1) 13/9/6 3:30:00 PM</td> <td>0.3</td> <td>2.2</td> </tr> <tr> <td>2) 13/9/9 11:45:00 AM</td> <td>0.6</td> <td>-</td> </tr> <tr> <td>3) 13/9/11 9:15:00 AM</td> <td>0.5</td> <td>-</td> </tr> </tbody> </table>				Date (dd/mm/yy)/Time	Water Depth (m)	Cave In (m)	1) 13/9/6 3:30:00 PM	0.3	2.2	2) 13/9/9 11:45:00 AM	0.6	-	3) 13/9/11 9:15:00 AM	0.5	-
Date (dd/mm/yy)/Time	Water Depth (m)	Cave In (m)																										
1) 13/9/6 3:30:00 PM	0.3	2.2																										
2) 13/9/9 11:45:00 AM	0.6	-																										
3) 13/9/11 9:15:00 AM	0.5	-																										

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



METRIC

RECORD OF BOREHOLE NO. 10



REFERENCE 12/09/12182 DATUM Geodetic LOCATION N 5209845.6 E 250256.3 - Muldrew Township Station 15+684 ORIGINATED BY JL
 PROJECT GWP 5468-09-00, Elboga Lake Culvert BOREHOLE TYPE Track Mounted CME 45B - NW Casing, NQ Core COMPILED BY AT
 CLIENT AECOM Inc. DATE (Started) 2013 September 9 TIME
 DATE (Completed) 2013 September 10 (Completed) CHECKED BY MAM

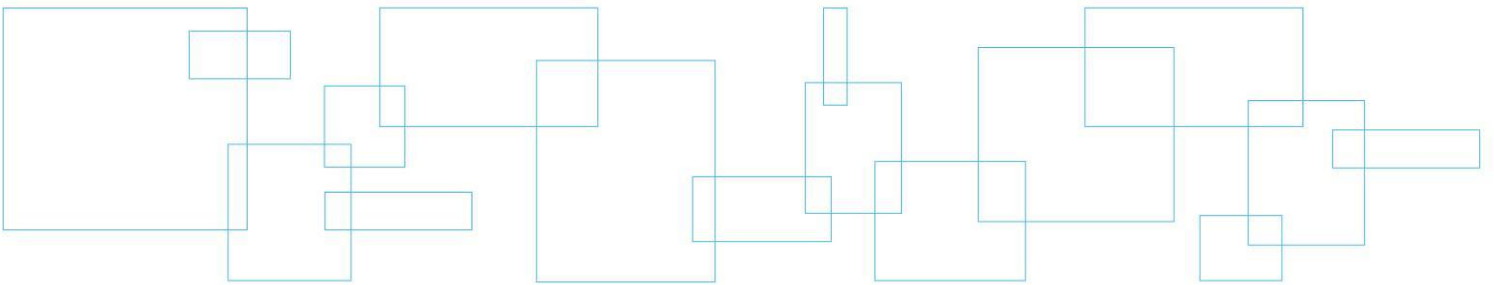
SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE			"N" VALUES	20	40	60	80	100	W _p	W		
431.6	Ground Surface															
0.0	50 mm forest mat (organics)		1	SS	19/250 mm											20 31 (49)
	SAND - cobble/boulder in sand and gravel matrix		2	SS	25/0 mm											
	(dense)		3	SS	25/0 mm											
429.2	BEDROCK - grey granitic rock		4	SS	25/50 mm											
2.4	fair to good quality		5	RC	Rec=100% RQD=71%											
			6	RC	Rec=97% RQD=82%											
426.2	End of Borehole															
5.4																
COMMENTS						+ 3, x 3 : Numbers on right refer to Sensitivity Numbers on left refer to values greater than 120 kPa O 3% STRAIN AT FAILURE										
						WATER LEVEL RECORDS										
						Date (dd/mm/yy)/Time					Water Depth (m)			Cave In (m)		
						1) 13/9/10 9:45:00 AM					2.7			3.7		
The stratification lines represent approximate boundaries. The transition may be gradual.						2)					-					
						3)					-					

MEL-GEO 12182 - BOREHOLE LOGS.GPJ MEL-GEO.GDT 14/3/17



Appendix 3 Borehole Plan and Lab Data

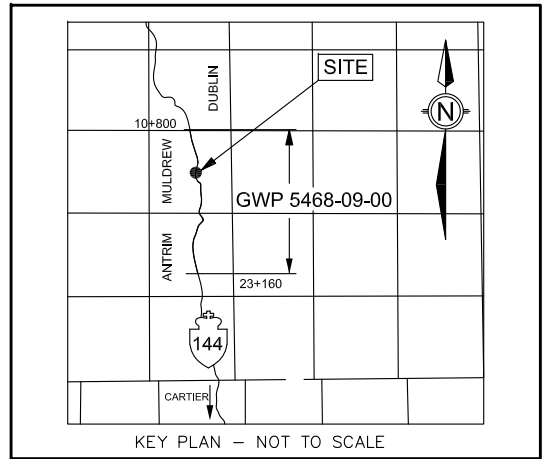
Drawing No. 2: Borehole Location and Soil Strata
Figure Nos. L-1 and L-3: Grain Size Distribution Curves
Figure No. L-4: Lab Test Summary Sheet



HWY 144
ELBOGA LAKE CULVERT & DETOUR
MULDREW TOWNSHIP

BOREHOLE LOCATIONS & SOIL STRATA

LVM



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

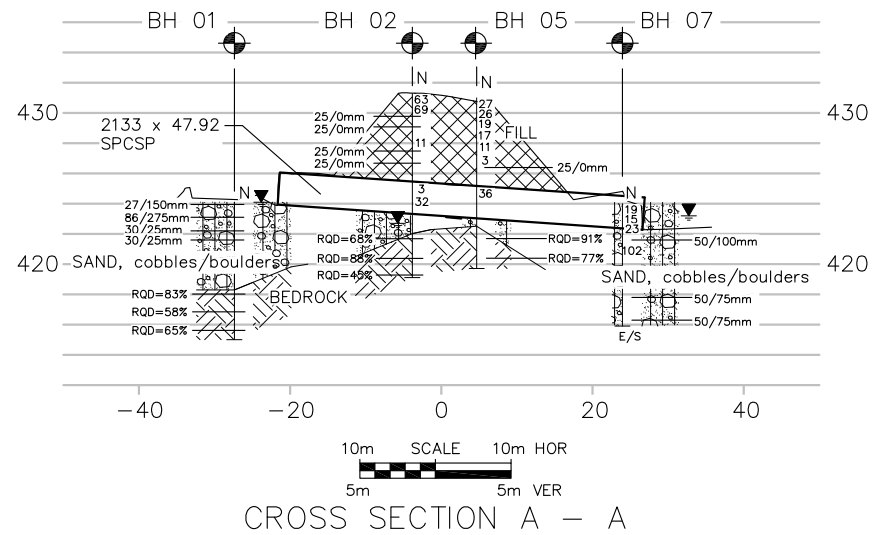
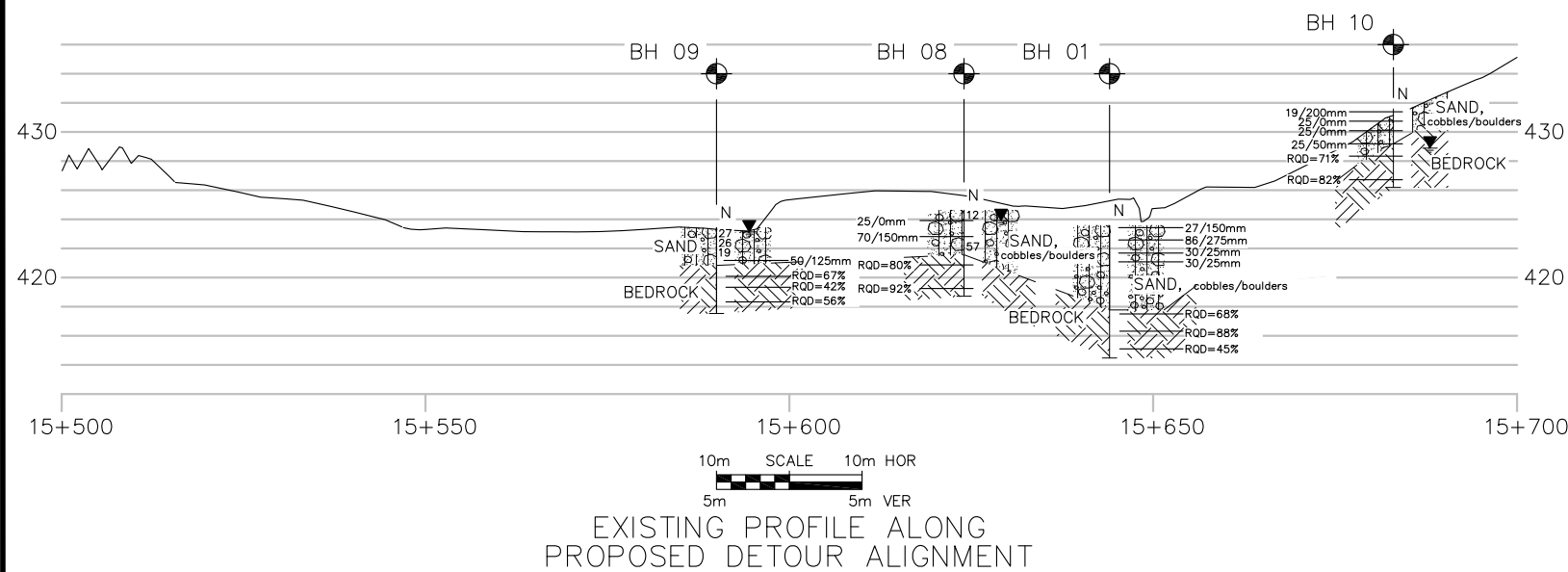
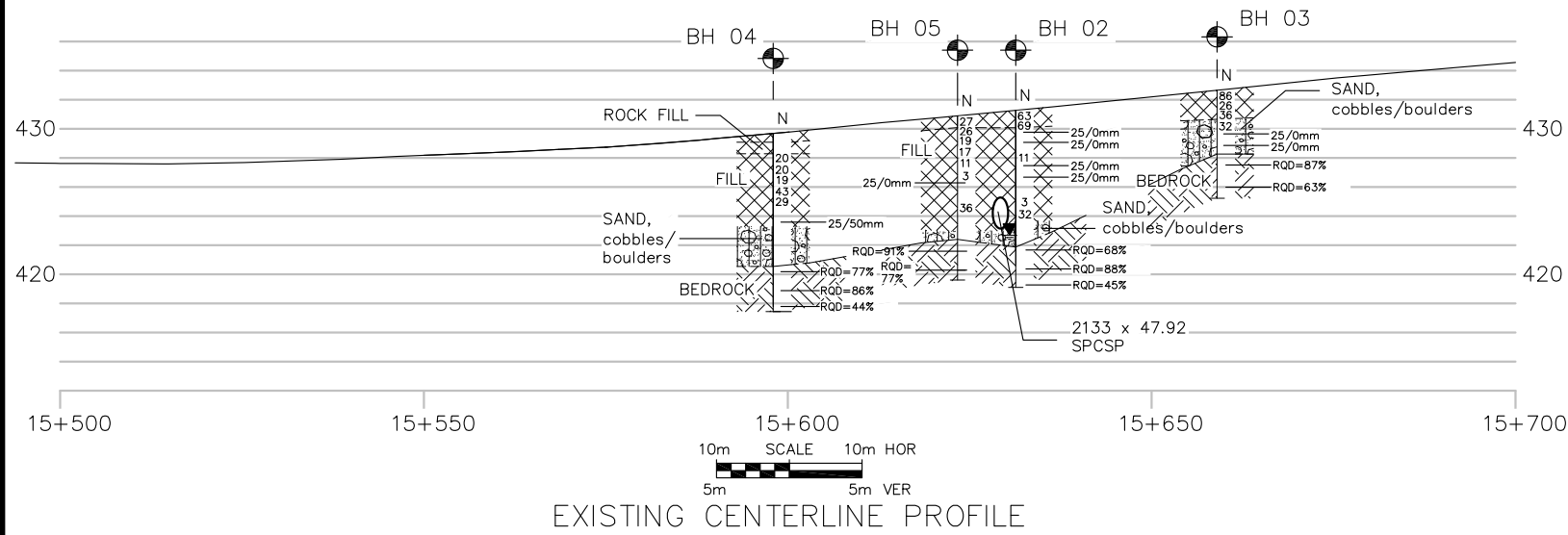
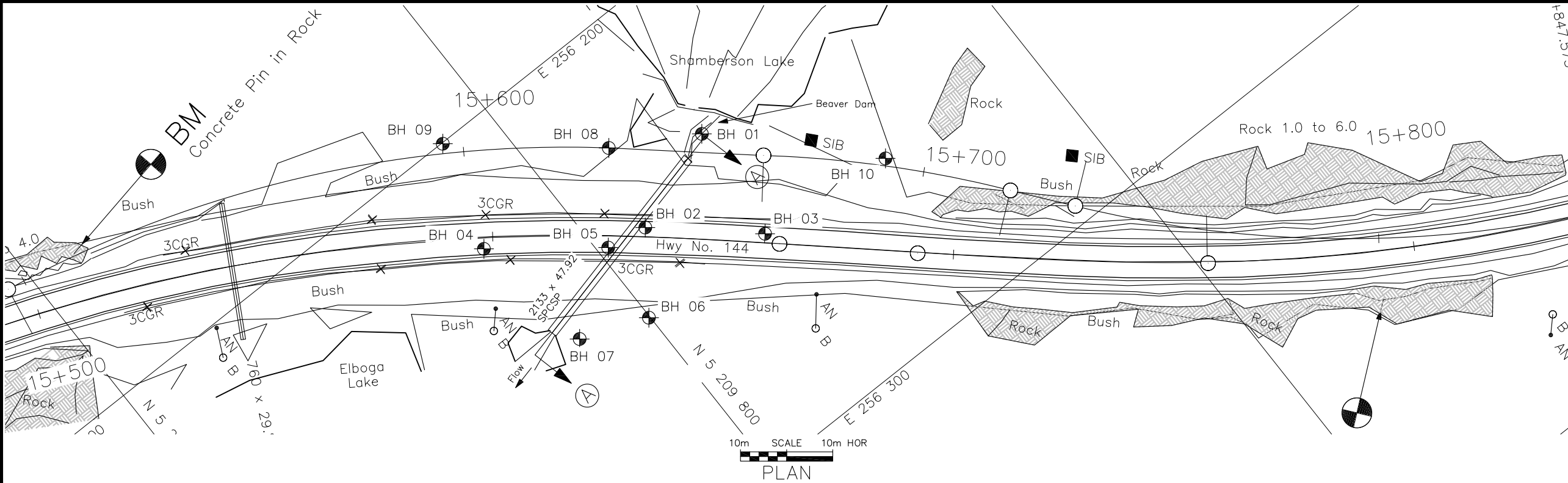
Borehole No.	Elev.	O/S	Co-ordinates	
			Northerly	Easterly
Borehole No. 1	424.1	23.0 m Lt	5209833.4	256233.4
Borehole No. 2	431.4	2.0 m Lt	5209811.2	256241.7
Borehole No. 3	432.7	2.0 m Lt	5209830.8	256258.9
Borehole No. 4	429.7	2.5 m Rt	5209780.9	256223.6
Borehole No. 5	430.9	2.6 m Rt	5209840.1	256202.2
Borehole No. 6	424.9	17.6 m Rt	5209799.7	256257.5
Borehole No. 7	424.0	22.5 m Rt	5209785.1	256251.8
Borehole No. 8	424.6	19.0 m Lt	5209815.7	256223.3
Borehole No. 9	423.4	21.0 m Lt	5209788.1	256200.2
Borehole No. 10	431.6	19.8 m Lt	5209845.6	250256.3

NOTE 1: 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.

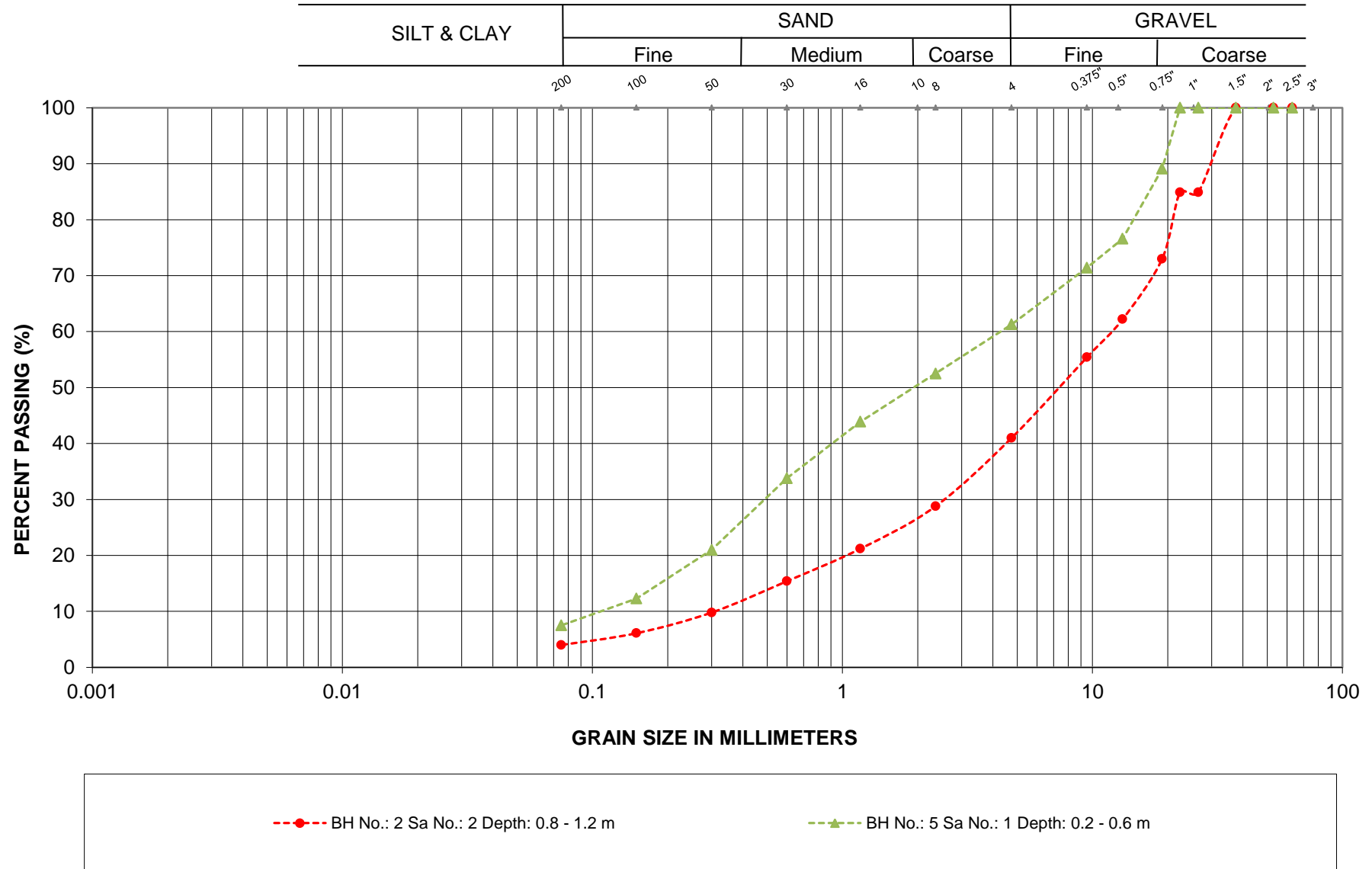
NOTE 2: The boundaries between soil strata have been established at the borehole locations only. The boundaries illustrated and stratigraphy between boreholes on this drawing are assumed based on borehole data and may vary. They are intended for design only.

REVISIONS	DATE	BY	DESCRIPTION
	DEC 2013	IK	DRAFT
	MAR 2014	RG	FINAL

HWY NO. 144 – MULDREW TWP.		
GEOCRES NO.: 41P-55		
LVM REFERENCE NO.: 12/09/12182		
DRAWN: IK	CHECKED: AT	DATE: December 2013



GRAIN SIZE ANALYSIS



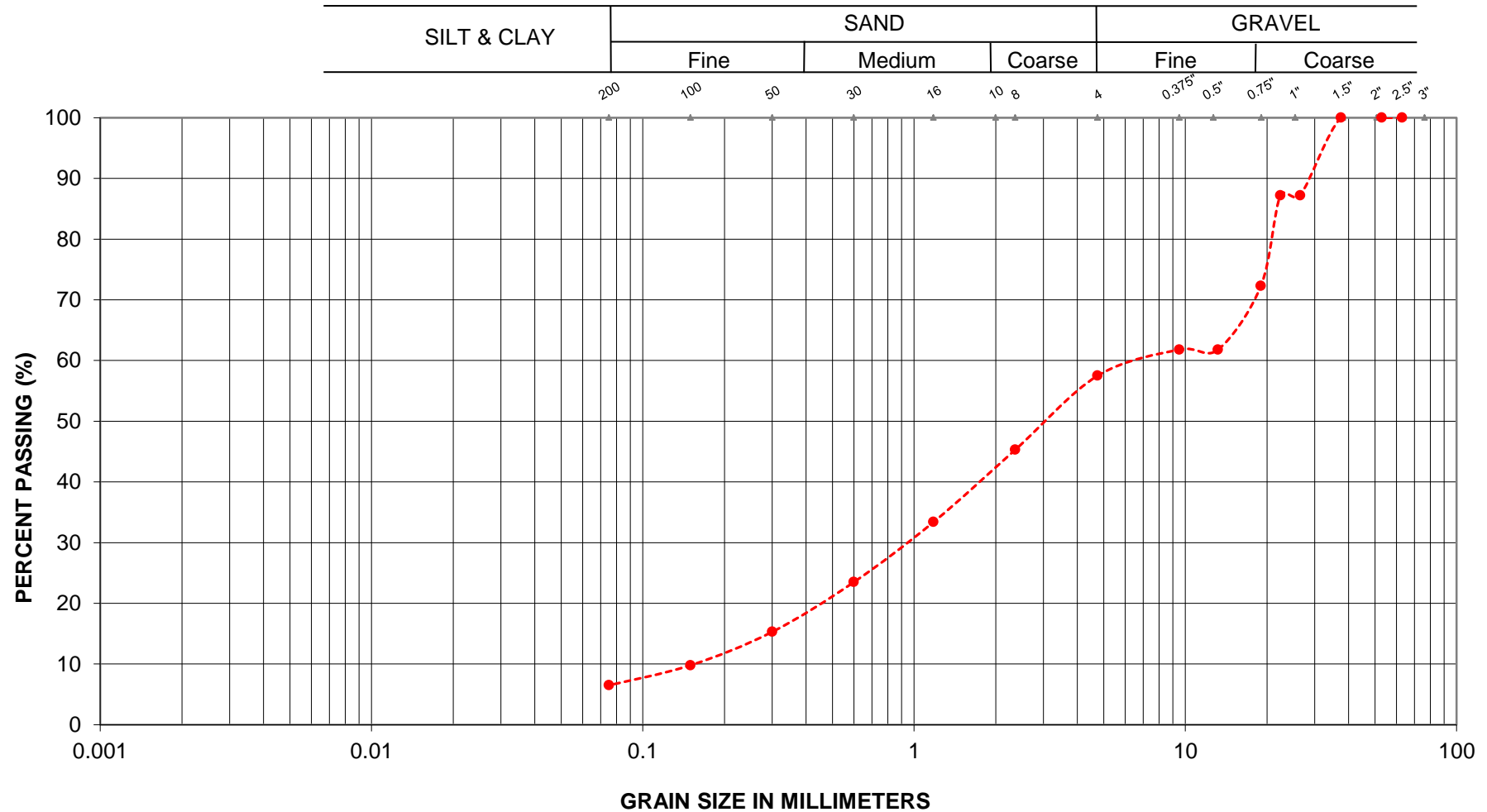
LOCATION: Hwy 144, Elboga Lake Culvert

GRANULAR FILL

LVM | MERLEX

FIGURE L-1

GRAIN SIZE ANALYSIS

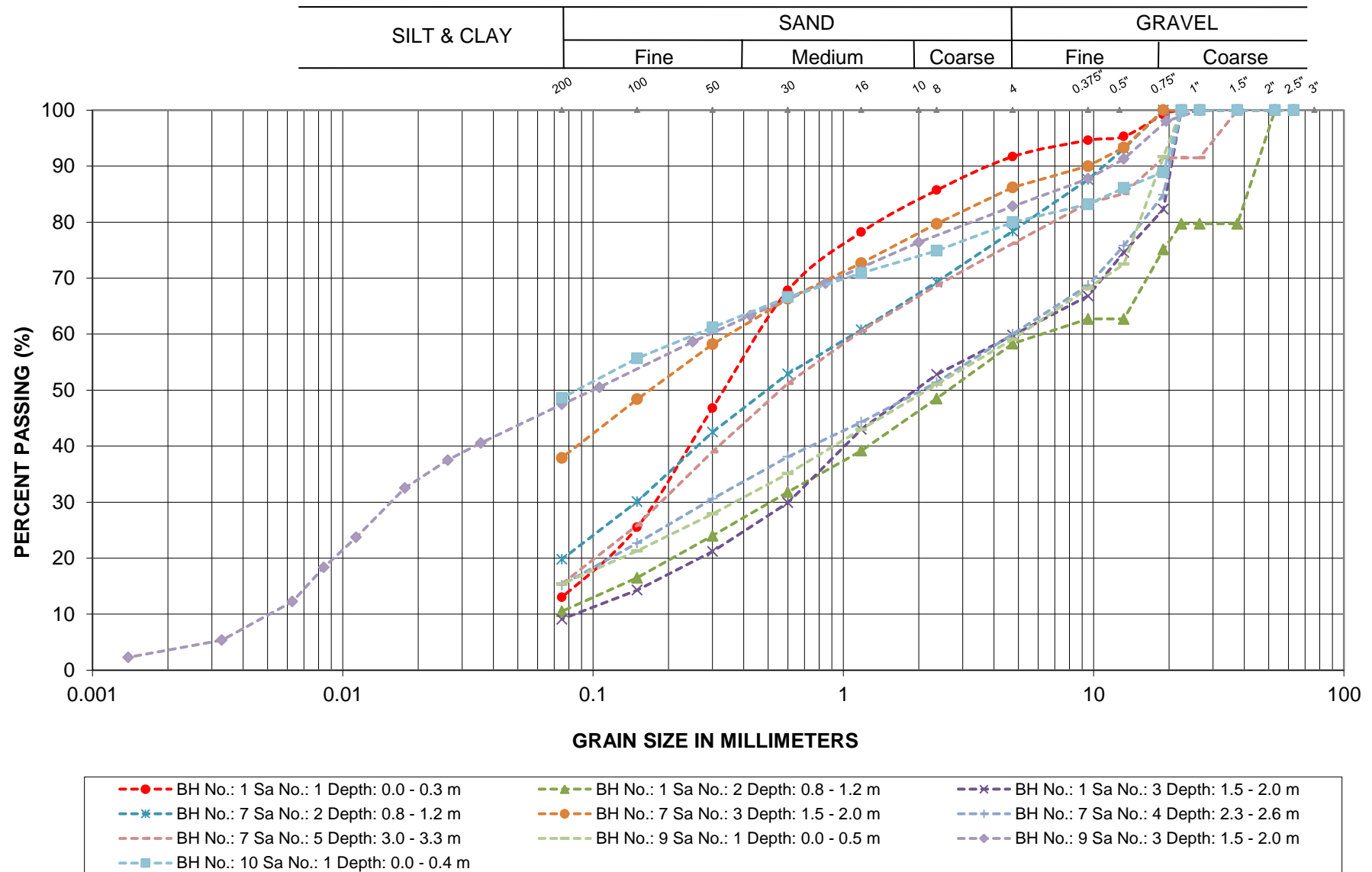


---●--- BH No.: 2 Sa No.: 5 Depth: 6.9 - 7.3 m

LOCATION: Hwy 144, Elboga Lake Culvert

FILL

GRAIN SIZE ANALYSIS



LOCATION: Hwy 144, Elboga Lake Culvert

SANDS

LVM | MERLEX

FIGURE L-3

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	8	79	13		34.4				27/150 mm			Sample contains organics
	2	0.8	42	47	11		8.8				86/275 mm			
	3	1.5	40	51	9		8.3				30/25 mm			
	4	2.3									80/225 mm			Recovered 1 gravel piece
	5	2.8									RC			Core - rock pieces
	6	3.4									RC			Core - rock pieces
	7	4.9									RC			Core - rock pieces
	8	5.8									RC			Bedrock - Rec = 100%
	9	6.4									RC			Bedrock - Rec = 100%
	10	7.9									RC			Bedrock - Rec = 100%
2	1	0.2					7.4				63			
	2	0.8	59	37	4		7.2				69			
	3	1.5									25/0mm			No recovery
	4	2.3									25/0mm			No recovery
	5	3.1									11			No recovery
	6	3.8									25/0mm			No recovery
	7	4.6									25/0mm			No recovery
	8	6.1									3			No recovery
	9	6.9	43	50	7		9.0				32			
	10	7.6									RC			Core - rock pieces
	11	8.8					12.5				50/100 mm			
	12	9.07									RC			Bedrock - Rec = 92%
	13	10.06									RC			Bedrock - Rec = 100%
	14	11.6									RC			Bedrock - Rec = 100%

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 (%)				
3	1	0.2					10.5				86			
	2	0.8					14.9				26			
	3	1.5									36			
	4	2.3									32			
	5	3.1									25/0mm			No recovery
	6	3.8									25/0mm			No recovery
	7	4.4									RC			Bedrock - Rec = 100%
	8	5.9									RC			Bedrock - Rec = 100%
4	1	0.2					7.4				85/250 mm			
	2	1.5									20			No recovery
	3	2.3									20			Recovered 1 gravel piece
	4	3.1									19			Recovered 3 gravel pieces
	5	3.8									43			Recovered 2 gravel pieces
	6	4.57									29			No recovery
	7	6.1									25/50 mm			No recovery
	8	6.43									RC			Core - rock pieces
	9	9.14									RC			Bedrock - Rec = 93%
	10	9.91									RC			Bedrock - Rec = 100%
	11	11.48									RC			Bedrock - Rec = 100%
5	1	0.15	39	53	8		11.25				27			
	2	0.76					10.11				26			
	3	1.52									19			Recovered 9 gravel pieces
	4	2.29									17			Recovered 1 gravel piece
	5	3.05									11			Recovered 5 gravel pieces
	6	3.81									3			No recovery

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 (%)				
	7	4.6									25/0mm			No recovery
	8	6.1									36			No recovery
	9	8.1									50/100 mm			Gravel pieces with sand
	10	8.2									RC			Core - rock pieces
	11	8.5									RC			Bedrock - Rec = 100%
	12	10.1									RC			Bedrock - Rec = 100%
6	1	0.0					21.0				16			Sample contains organics
	2	0.8					18.7				25			Sample contains organics
	3	1.5					9.5				73			
	4	2.3									25/0mm			No recovery
	5	3.1									25/0mm			No recovery
	6	3.8									25/0mm			No recovery
	7	4.6									25/0mm			No recovery
	8	4.9									RC			Bedrock - Rec = 60%
	9	6.4									RC			Bedrock - Rec = 100%
7	1	0					9.63				12			
	2	0.76	22	58	20		9.7				15			
	3	1.52	14	48	38		22.12				23			
	4	2.29	40	45	15		14.24				50/100 mm			
	5	3.05	24	61	15		10.1				102			
	6	3.81									25/0mm			No recovery
	7	4.57									25/0mm			No recovery
	8	6.1					17.3				50/75 mm			
	9	7.62					9.49				50/75 mm			

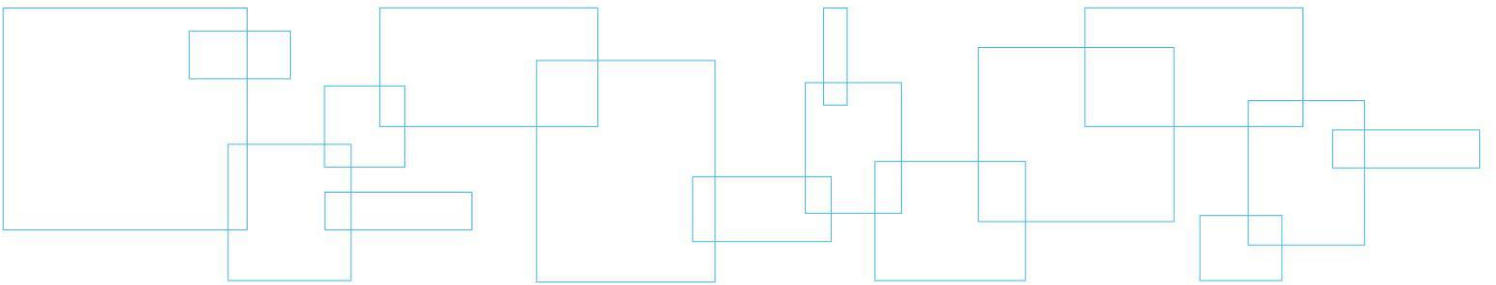
Laboratory Tests - Summary Sheet

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 (%)				
8	1	0					5.64				12			
	2	0.76									25/0 mm			No recovery
	3	1.52					8.57				70/150 mm			
	4	2.29					10.69				57			
	5	3.1									RC			Bedrock - Rec = 100%
	6	4.6									RC			Bedrock - Rec = 100%
9	1	0	41	44		15	12.08				15			
	2	0.76					12.71				36			
	3	1.52	18	35	43	4	13.95				47			
	4	2.29					11.08				50/125 mm			
	5	2.6									RC			Bedrock - Rec = 100%
	6	4.1									RC			Bedrock - Rec = 100%
	7	4.7									RC			Bedrock - Rec = 98%
10	1	0	20	31		49	14.48				19/250 mm			
	2	0.76									25/0 mm			No recovery
	3	1.52									25/0 mm			No recovery
	4	2.29					15.89				25/50 mm			
	5	2.4									RC			Bedrock - Rec = 100%
	6	3.9									RC			Bedrock - Rec = 97%

Appendix 4 Photo Essay

Enclosure No. 12:

Photo Essay



West End of Culvert – Looking South

Photo: 1



View of Stream at Inlet – Looking West, Note Beaver Dam

Photo: 2



Project: Hwy 144 – Elboga Lake Culvert

Photos Provided By: LVM

Date: September 2013

View through Culvert – Looking East

Photo: 3



East End of Culvert – Looking South

Photo: 4



Project: Hwy 144 – Elboga Lake Culvert

Photos Provided By: LVM

Date: September 2013

East Embankment Slope – Looking North

Photo: 3



Project: Hwy 144 – Elboga Lake Culvert

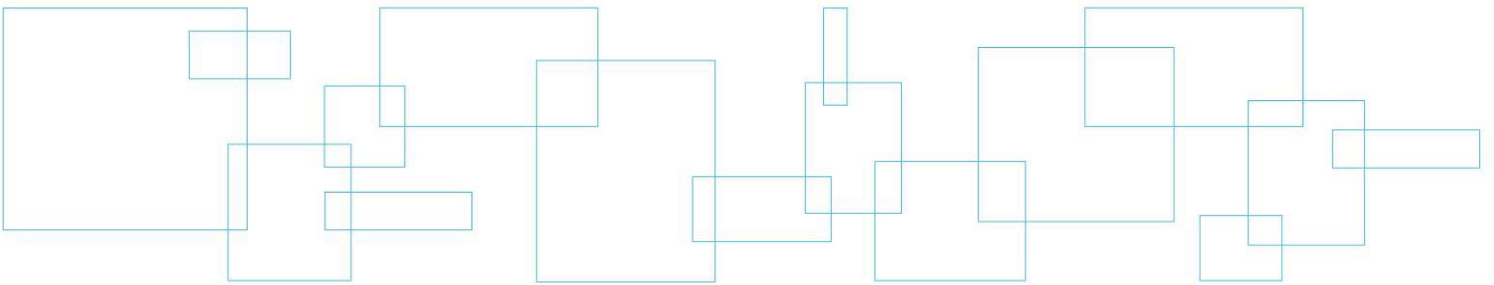
Photos Provided By: LVM

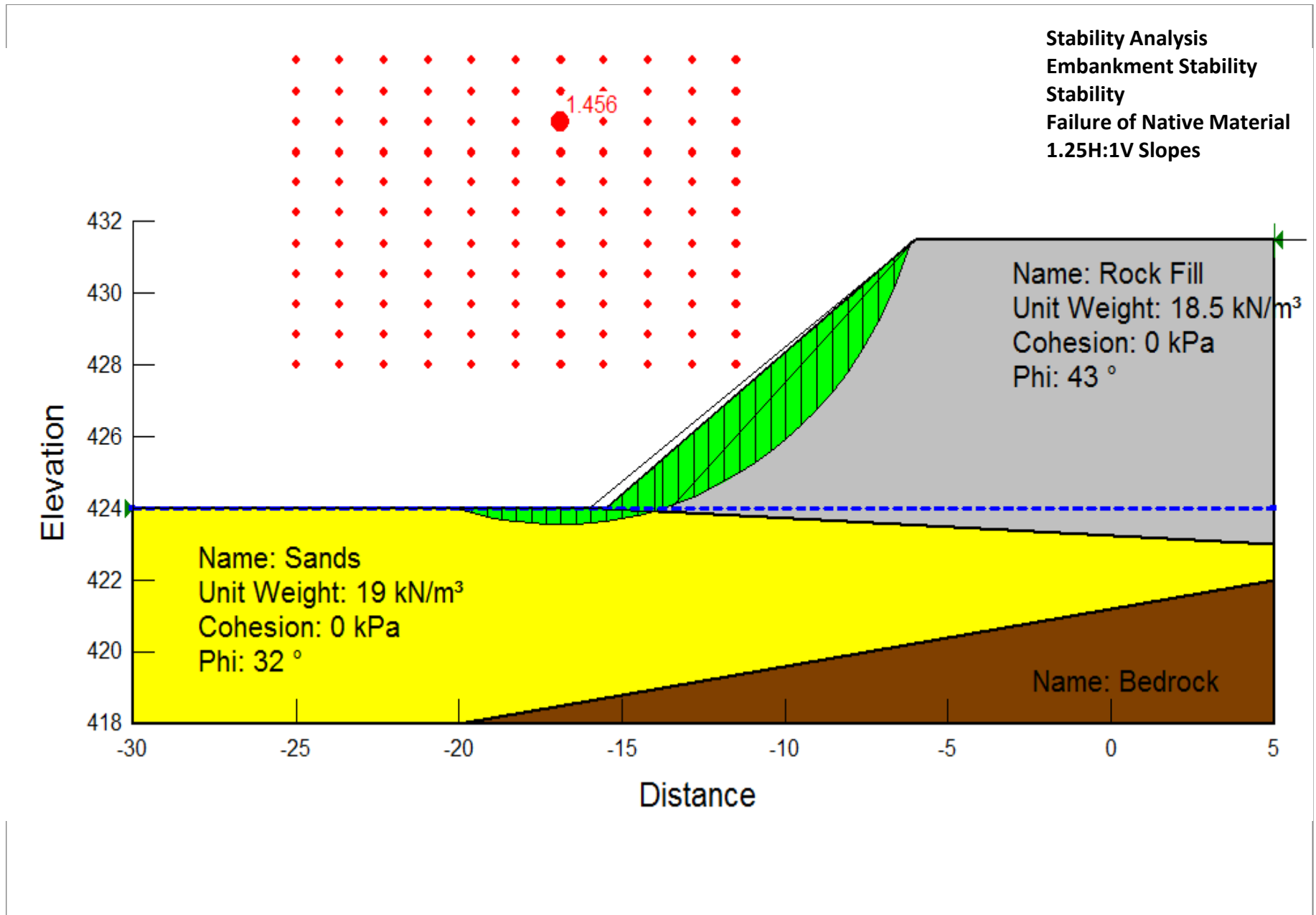
Date: September 2013

Appendix 5 Design Data

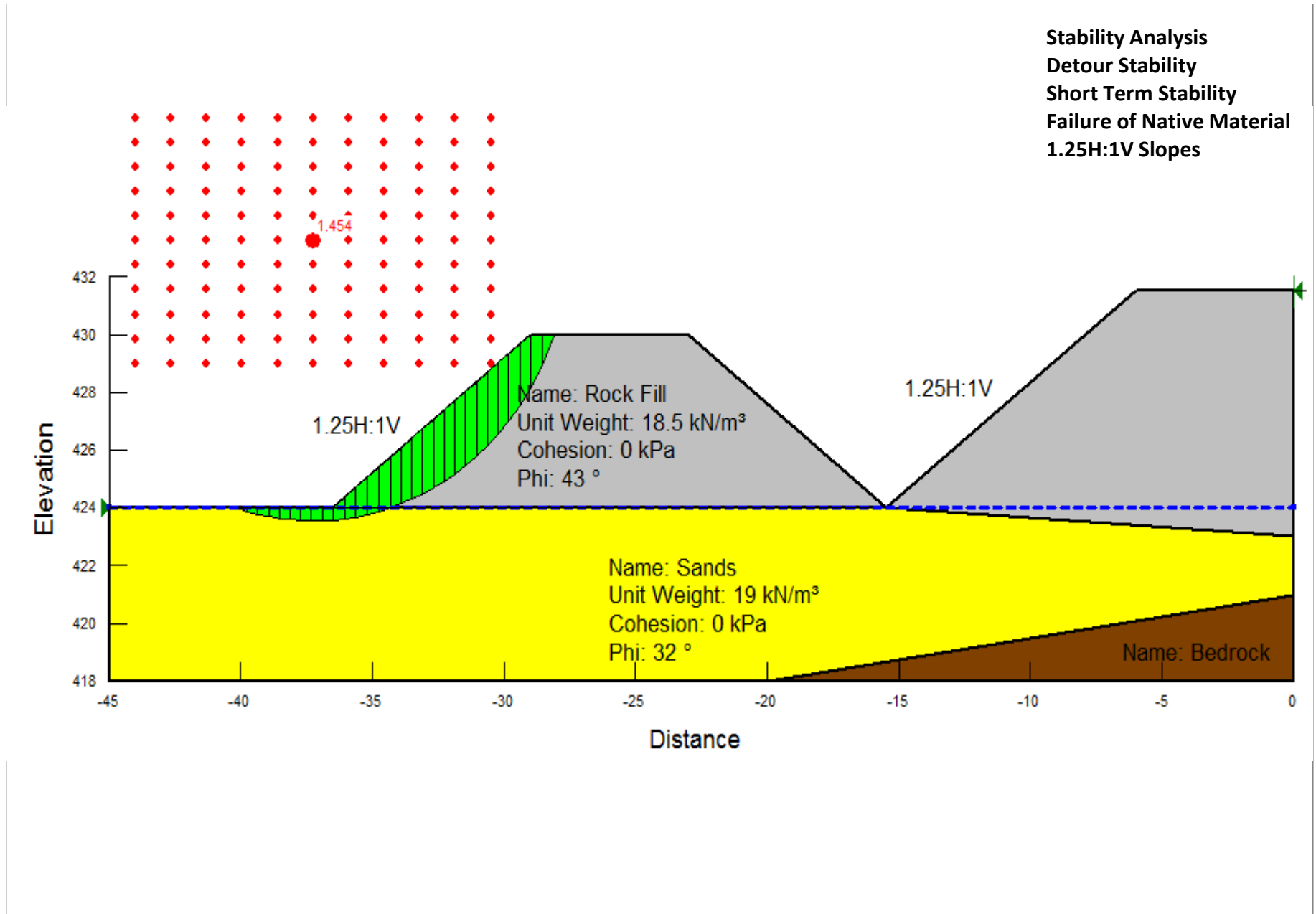
Figure Nos. S-1 and S-2:

Slope Stability





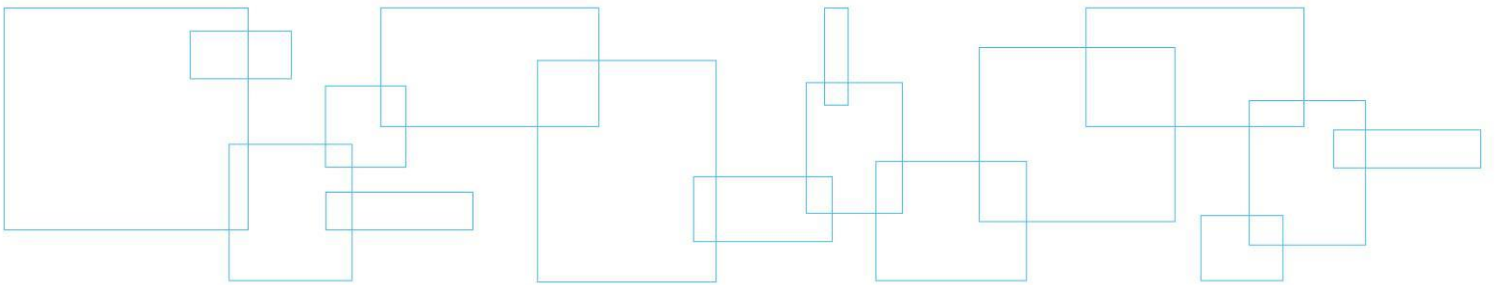
Stability Analysis
Station 15+629
TWP of Muldrew



Stability Analysis
Station 15+629
TWP of Muldrew

Appendix 6 Notice to Contractor

Notice to Contractor



NOTICE TO CONTRACTOR – Obstructions in Embankment and Native Soils

Special Provision

The Contractor is advised that rock fill is present throughout the embankment fill material and cobble and boulder size rock was present through the native dense sand and gravel subgrade. The contractor should be prepared to deal with the large rock sizes and dense soils.