



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

**Highway 65 Rehabilitation
Culvert Replacement
Station 11+814 - Twp. of Dymond
GWP 5574-04-00**

**Highway 65
From 0.1 km East of Armstrong Street, Easterly 22.5 km to the Ontario/Quebec
Boundary**

FINAL FOUNDATION INVESTIGATION AND DESIGN REPORT

Date: November 21, 2013
Ref. N^o: 12/03/12028-F1

Geocres No. 31M-104

LVM | MERLEX



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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 an existing centreline culvert site. The site is located on Highway 65, some 1.8 km East of Highway 11, in the Township of Dymond.

The foundation investigation location was specified by the MTO in the Terms of Reference for extra work under Agreement No. 5010-E-0028. The terms of reference for the scope of work are outlined in LVM | MERLEX's Proposal P-11-023, dated August, 2012. 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 foundation investigation was undertaken for the Structural Plate Corrugated Steel Pipe (SPCSP) culvert is located on Highway 65 at Station 11+814, Township of Dymond. The topography at the site is a low shallow slope valley area to the left and right of the embankment. The existing highway embankment currently supports two undivided lanes of highway, running in an east-west direction. The existing highway, at the culvert location, is constructed on an earth fill embankment some 7.5 m in height, with centerline elevation of 193.9 m at the culvert location. The culvert at this location is a 2.3 m diameter SPCSP culvert, some 61 m in length. Flow through the culvert is from north to south (left to right) (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 Temiskaming Clay Plain. The topography on this section of Highway 65 is generally flat. Significant layers of earth overlie the bedrock. Organic terrain was also observed. Within the project area native overburden consists primarily of a deep deposit of clays.

Bedrock in the area, as indicated on OGS Map 2506, is of the Middle/Late Silurian. At the location of this culvert foundation investigation, the bedrock comprises of dolostone, limestone, sandstone, and shale.

2.2 HISTORICAL INFORMATION

In 1961, a DHO Foundation Report (Geocres No. 31M-009) was prepared for a grade raise and culvert replacement at Station 59+35 in the Township of Dymond (approximately Station 11+810). At that time the report indicates that the embankment was about 4.3 m (14.0') in height, and was to be raised by some 2.4 m (8.0'). This report indicated the embankment consisted of rock fill. The new culvert was recommended to be a flexible type culvert, installed on a camber, to accommodate settlement. New berms (some 3.1 m (10') in length) were recommended for embankment stability.

3 INVESTIGATION PROCEDURES

The field work for this investigation was carried out during the period of November 5th to 16th, 2012 during which time six (6) sampled boreholes and DCPTs, were advanced. Four (4) boreholes were advanced through the embankment up and down chainage from 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 and a truck 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. At select boreholes, a Dynamic Cone Penetration Test (DCPT) was carried out to give a continuous plot of the soil resistance with depth. When cohesive deposits were encountered, the in-situ strength was measured using an “N” size field vane, vane collar, and calibrated torque meter. All samples taken during this investigation were stored in labeled airtight containers for transport to our North Bay laboratory for visual examination and select laboratory testing.

Groundwater conditions in the open boreholes were observed during the advancement of and immediately following, completion of the individual boreholes. 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 asphalt surface.

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, particle size analysis, Atterberg Limits determination, as well as specific gravity testing. Consolidation testing was also carried out on two samples of the native clay deposit. 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-9).

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 11+814 was described as a nail and washer in the east face of Hydro Pole at Station 11+833.6, 22. 6 m left of centerline (see Drawing No. 2,

Appendix 3). The elevations are derived from the Geodetic Benchmark 011982U080 described as the Brass Tablet set in the concrete foundation of a livestock barn at Station 13+167.2, 60.7 m right of centerline.

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 and Dynamic Cone Penetration Test (DCPT), plus field observations. Typically such boundaries represent transitions from one zone to another and are not an exact demarcation of specific geological unit. Additional consideration should be given to the fact that subsurface conditions may vary markedly between adjacent boreholes and beyond any specific boring location, and are shown on the drawings for illustration purposes only.

4.1 CULVERT STATION 11+814, TWP OF DYMOND

A plan and profile illustrating the borehole locations and stratigraphic sequences is shown on Figure No. 2, Appendix 3. During the course of the exploration program, six (6) sampled boreholes were put down at this site, with Borehole Nos. 1 and 6 advanced at the culvert ends (left and right, respectively), and Borehole Nos. 2 to 5 advanced through the embankment. At the time of the subsurface investigation, the ground surface elevations at Boreholes Nos. 1 to 6 were recorded at 187.9, 193.0, 192.8, 193.3, 192.8, and 187.6 m, respectively.

4.1.1 Pavement Structure

At surface at Borehole Nos. 2 and 3, a pavement structure consisting of 75 mm of asphalt and 150 mm crushed gravel was penetrated. At surface at Borehole Nos. 4 and 5, a layer of crushed gravel some 100 mm thick was penetrated.

4.1.2 Granular Fill

Underlying the pavement structure at Borehole Nos. 2 to 5, a deposit of granular fill consisting of brown sand trace silt trace gravel was penetrated. The natural moisture content measured on samples of this deposit was in the order of 4 to 14%. Gradation analyses were carried out on two (2) samples of this deposit, the results of which indicated 4 to 30% gravel size particles, 58 to 89% sand size particles, and 7 to 12% silt and clay size particles (Figure No. L-1, Appendix 3). Based on SPT 'N' values of 12 to 26 blows per 300 mm penetration, the compactness of this deposit was described as compact. This deposit was encountered to depths of 1.8, 2.1, 1.1, and 1.4 m below grade at Borehole Nos. 2 to 5, respectively (elevations 191.2, 190.7, 192.2, and 191.4 m, respectively).

4.1.3 Silty Clay Fill

Underlying the granular fill at Borehole No. 2 a deposit of grey silty clay fill was penetrated. Trace asphalt (i.e. a 25 mm thick layer) was encountered in this deposit at elevation 190.5 m. The natural moisture content measured on samples of this fill deposit was in the order of 16 to 32%. This deposit was encountered to a depth of 3.5 m below grade (elevation 189.5 m).

4.1.4 Sand and Clay Fill

Underlying the silty clay fill at Borehole No. 2, and underlying granular fill at Borehole Nos. 3 and 4, a deposit of fill described as grey sand and clay some silt some gravel was penetrated. The natural moisture content measured on samples of this deposit was in the order of 4 to 30%. A gradation analysis was carried out on one (1) sample of this deposit, the results of which indicated 20% gravel size particles, 35% sand size particles, 17% silt size particles, and 28% clay size particles (Figure No. L-2, Appendix 3). Atterberg Limits testing was carried out on the clay portion of one (1) sample of this deposit, the results of which indicated a Plastic Limit in the order of 18% and a Liquid Limit in the order of 47% (Figure No. L-2, Appendix 4). Based on the results of the Atterberg Limits testing, this deposit was described as a silty clay of medium plasticity (CI). This deposit was encountered to depth of 7.3, 5.1, and 2.1 m below grade at Borehole Nos. 2, 3, and 4, respectively (elevations 185.7, 187.7, and 191.2 m, respectively).

4.1.5 Sand Fill

Underlying the sand and clay fill at BH No. 3, a deposit of sand fill consisting of grey sand trace silt trace gravel was penetrated. The natural moisture content measured on samples of this deposit was in the order of 18 to 20%. Based on SPT 'N' values of 9 to 20 blows per 300 mm penetration, the compactness of this deposit was described as loose to compact. This deposit was encountered to a depth of 7.3 m below grade (elevation 185.5 m).

4.1.6 Clay Fill

At surface at Borehole Nos. 1 and 6, a deposit of fill described as brown clay with silt was penetrated. Occasional cobbles and boulders were encountered in this deposit. The natural moisture content measured on samples of this deposit was in the order of 37 to 40%. This deposit was encountered to depth of 0.6 and 0.8 m below grade at Borehole Nos. 1 and 6, respectively (elevations 187.3 and 186.8 m, respectively).

4.1.7 Clay

Underlying the clay fill at Borehole Nos. 1 and 6, underlying the sand and clay fill at Borehole Nos. 2 and 4, underlying the sand fill at Borehole No. 3, and underlying the granular fill at Borehole No. 5, a deposit of grey clay some to with silt was penetrated. Trace organics and wood pieces were encountered in the upper portions of this deposit at Borehole Nos. 1, 2, 3, and 6. The natural moisture content measured on samples of this deposit was in the order of 35 to 63%. Hydrometer analyses were carried out on five (5) samples of this deposit, the results of which indicated 0% gravel size particles, 0 to 2% sand size particles, 12 to 26% silt size particles, and 69 to 88% clay size particles (Figure No. L-3, Appendix 3). Atterberg Limits testing was carried out on five (5) samples of this deposit, the results of which indicated a Plastic Limit in the order of 21 to 32% and a Liquid Limit in the order of 52 to 80% (Figure No. L-5, Appendix 4). Based on the results of the Atterberg Limits testing, this deposit was described as a clay of high plasticity (CH). Based on in-situ shear strengths of 30 kPa to greater than 100 kPa, the consistency of this deposit was described as firm to very stiff, generally stiff (Figure No. L-6, Appendix 3). This deposit was encountered to depths of 3.8, 10.7, 10.6, 9.1, 7.6, and 3.0 m below grade at Borehole Nos. 1 to 6, respectively, where a transition to a varved clay was observed (elevations 184.1, 182.3, 182.2, 184.2, 185.2, and 184.6 m, respectively).

One (1) one-dimensional oedometer (consolidation) test was carried out on a sample of the clay deposit (Borehole No. 1, Sample 7). The preconsolidation pressure was estimated (using the Casagrande method) to be in the order 85 kPa. The over-consolidation ratio, which is the ratio of the preconsolidation pressure to the existing effective overburden pressure, was in the order of 2.2. Based on the results of the oedometer (consolidation) tests, vane shear strength data, and the relationship of the moisture content to liquid limit, this deposit is considered to be slightly overconsolidated, relative to the existing overburden pressure. Results from the consolidation tests are shown on enclosed Figure No. L-7, Appendix 3.

4.1.8 Varved Clay

Underlying the clay at Borehole Nos. 1 to 6, a deposit of grey clay some to with silt was penetrated. Silty clay varves were encountered in this deposit. This deposit consisted of clay layers some 25 mm thick interbedded with silty clay varves some 6 mm thick. The natural moisture content measured on samples of this deposit was in the order of 32 to 67%. The natural moisture content measured on a sample of a silty clay varve of this deposit was in the order of 36%. Hydrometer analyses were carried out on three (3) samples of this deposit, the results of which indicated 0% gravel size particles, 0% sand size particles, 18 to 40% silt size particles, and 60 to 82% clay size particles (Figure No. L-4, Appendix 3). Atterberg Limits testing was carried out on four (4) samples of the clay portion of this deposit, the results of which indicated a Plastic Limit in the order of 20 to 22% and a Liquid Limit in the order of 52 to 67% (Figure No. L-5, Appendix 4). Based on the results of the Atterberg Limits testing, the clay portion of this deposit was described as a clay of high plasticity (CH). Atterberg Limits testing was carried out on one (1) sample of the silty clay portion of this deposit, the results of which indicated a Plastic Limit in the order of 20% and a Liquid Limit in the order of 33% (Figure No. L-5, Appendix 4). Based on the results of the Atterberg Limits testing, the clay portion of this deposit was described as a silty clay of low plasticity (CL). Based on in-situ shear strengths of 30 to 56 kPa, the consistency of this deposit was described as firm to stiff (Figure No. L-6, Appendix 3). Sampling was terminated in this deposit at depths of 6.9, 14.5, 13.0, 9.9, 12.9, and 6.9 m below grade at Borehole Nos. 1 to 6, respectively (elevations 181.0, 178.5, 179.8, 183.4, 179.9, and 180.7 m, respectively).

One (1) one-dimensional oedometer (consolidation) test was carried out on a sample of the deposit of the clay with varves (Borehole No. 3, Sample 11). The preconsolidation pressure was estimated (using the Casagrande method) to be in the order 160 kPa. The over-consolidation ratio, which is the ratio of the preconsolidation pressure to the existing effective overburden pressure, was in the order of 1.4. Based on the results of the oedometer (consolidation) tests, vane shear strength data, and the relationship of the moisture content to liquid limit, this deposit is considered to be slightly overconsolidated, relative to the existing overburden pressure. Results from the consolidation tests are shown on enclosed Figure No. L-8, Appendix 3.

4.1.9 DCPT

Dynamic Cone Penetration Tests (DCPT) were advanced at Borehole Nos. 1 and 4. DCPT refusal was encountered at depths of 38.7 and 39.5 m below grade at Borehole Nos. 1 and 4 (elevations 149.2 and 153.8 m, respectively).

4.2 GROUNDWATER DATA

At the time of this investigation, the water level in the culvert was measured at elevation 187.3 m at the culvert outlet.

Measurements of the groundwater table and cave-in levels were undertaken, where possible, in the open boreholes during the advance of the individual borings and upon completion. Standpipes were installed in Borehole Nos. 1, 4, 5, and 6, to obtain post completion water levels. These levels are recorded on the individual Record of Borehole Log Sheets (Appendix B). The water levels in Borehole Nos. 1 to 6 were measured between elevations 184.5 to 188.3 m, respectively.

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

The results of the post completion water level monitoring are provided in the following table:

Table 1 Groundwater level measurements

DATE	BH NO. 1		BH NO. 4		BH NO. 5		BH NO. 6	
	Depth (m)	Elevation (m)	Depth (m)	Elevation (m)	Depth (m)	Elevation (m)	Depth (m)	Elevation (m)
Nov 5	Dry	-	-	-	-	-	-	-
Nov 6	1.5	186.4	-	-	-	-	-	-
Nov 7	1.0	186.9	-	-	-	-	5.2	182.4
Nov 8	0.8	187.1	-	-	-	-	1.8	185.8
Nov 9	0.8	187.1	Dry	-	-	-	1.0	186.6
Nov 12	0.8	187.1	8.6	185.3	Dry	-	1.0	186.6
Nov 13	0.8	187.1	8.6	185.3	8.9	183.9	1.0	186.6
Nov 14	0.8	187.1	8.6	185.3	8.5	184.3	1.0	186.6
Nov 15	0.8	187.1	8.2	185.7	8.3	184.5	1.0	186.6

5 DISCUSSION AND RECOMMENDATION

5.1 GENERAL

The Ministry of Transportation of Ontario (MTO) is planning to replace the culvert at Station 11+814 on Highway 65. The existing SPCSP culvert is 2.3 m in diameter and some 61 m in length. Flow through the culvert is from north to south (left to right). The invert of the culvert is at approximately elevation 186.3 m at highway centreline. Plans call for the culvert at Station 11+814 to be replaced by a 2700 mm diameter concrete pipe culvert since the bottom of the existing SPCSP has deteriorated due to the corrosive nature of the groundwater in this area. Currently it is not proposed to raise the grade nor widen the platform as part of the culvert replacement work at this location.

5.2 CULVERT REPLACEMENT

Three different methods for the replacement of the culvert at Station 11+814 are discussed in this report. These methods include temporary open excavations with single lane traffic staging to the north and south of the existing highway centreline, excavations using temporary centreline protection systems (sheet piling), and employing trenchless/micro-tunneling technologies for culvert replacement.

5.2.1 Temporary Open Excavation

Should temporary open excavations be used, a cut of some 7 m deep (from about elevation 193 to 186 m at the existing highway), perpendicular to the existing culvert alignment will be required in the existing driving lanes for the replacement of the culvert at Station 11+814. As the traffic has to be maintained during construction, a two stage staged operation is proposed. The staged construction, as provided by AECOM, is presented in Figures 1 and 2, below.

In Stage 1, a first cut with an angle of 1.5H:1V perpendicular to the driving lane will be carried out to permit replacement the north section of the culvert. For this stage, a temporary driving lane will be constructed south of the highway. A temporary embankment will be constructed over the existing embankment south of the highway in order to provide a temporary detour driving surface of 5 m wide at elevation 192 m. Following completion of the installation and backfilling of the concrete pipe on the north side of the highway during Stage 1, the temporary embankment will be removed and the north section of the embankment will be reinstated and Stage 2 will be carried out to replace the south section of the culvert. Stage 2 will be carried out in a similar manner to Stage 1.

The replacement of the culvert shall be carried out as per OPSS 421 and OPSD 803.010.

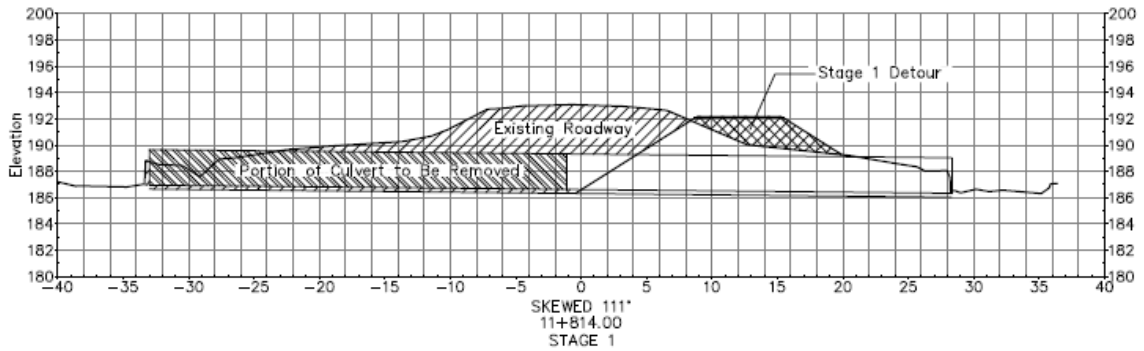


Figure 1: Staged construction for the replacement of culvert at Station 11+814, Stage 1

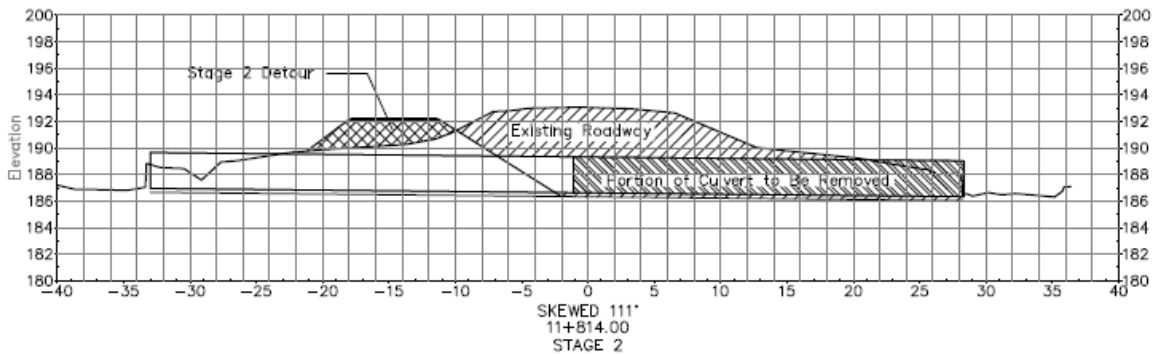


Figure 2: Staged construction for the replacement of culvert at Station 11+814, Stage 2

A stability analysis, using the GEO-SLOPE computer program, Slope/W (GeoStudio 2007, version 7.17, Geo-Slope International Ltd.), was carried out at the culvert location (Station 11+814) with the geometry shown on Figures 3 to 6. Two types of analyses were carried out: static short term analyses and static long term analyses. A safety factor of 1.3 is required to assure the stability of the temporary open excavation. Table 2 below presents the soil parameters used in these analyses, which were based on the soil profile shown on Boreholes Nos. 1 to 6. The water table was considered at an elevation of 185 m, which corresponds to the elevation of the adjacent stream in a dewatered condition during excavation. A uniform surcharge of 17 kN/m^2 , corresponding to the wheel loads, has been applied on the temporary driving surface.

Table 2: Soil parameters used in the stability analyses

PARAMETER	MATERIAL				
	GRANULAR EMBANKMENT	SAND WITH GRAVEL, SOME SILT (FILL)	CLAY WITH SILT TO SAND AND CLEY (FILL)	CLAYEY DEPOSIT	
				SHORT TERM	LONG TERM
Unit weight, γ (kN/m^3)	20	20	17	17	17

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CULVERT STATION 11+814, TWP OF DYMOND

Submerged unit weight, γ' (kN/m ³)	10	10	7	7	7
Effective cohesion, c' (kPa)	0	0	7	---	7
Effective friction angle ϕ' (°)	30	32	30	0	30
Undrained shear strength, S_u (kPa)	---	---		40	---

Results of the stability analyses are presented in Table 3 below. The cross sections, the slip surfaces and, the factors of safety are presented below on Figures 3 to 6.

Table 3: Results of the stability analyses – Perpendicular to the road center line

CROSS SECTION	FACTOR OF SAFETY (F.S.)		
	MINIMUM F.S. CALCULATED		MINIMUM F.S. REQUIRED
	SHORT TERM	LONG TERM	
Stage 1	1.29	1.29	1.3
Stage 2	1.28	1.28	

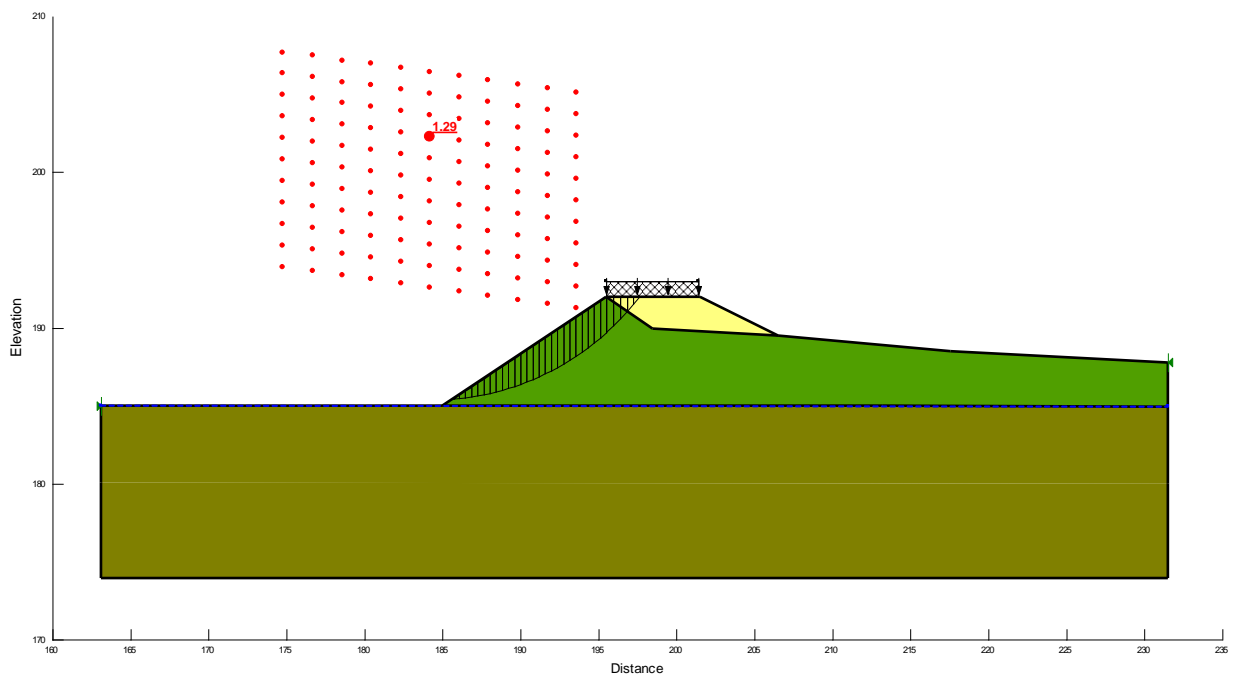


Figure 3: Short term stability analyses - Stage 1- Perpendicular to the road center line

Legend – Figures 3 to 6	
	Fill : Granular embankment
	Fill : Sand with gravel and some silt

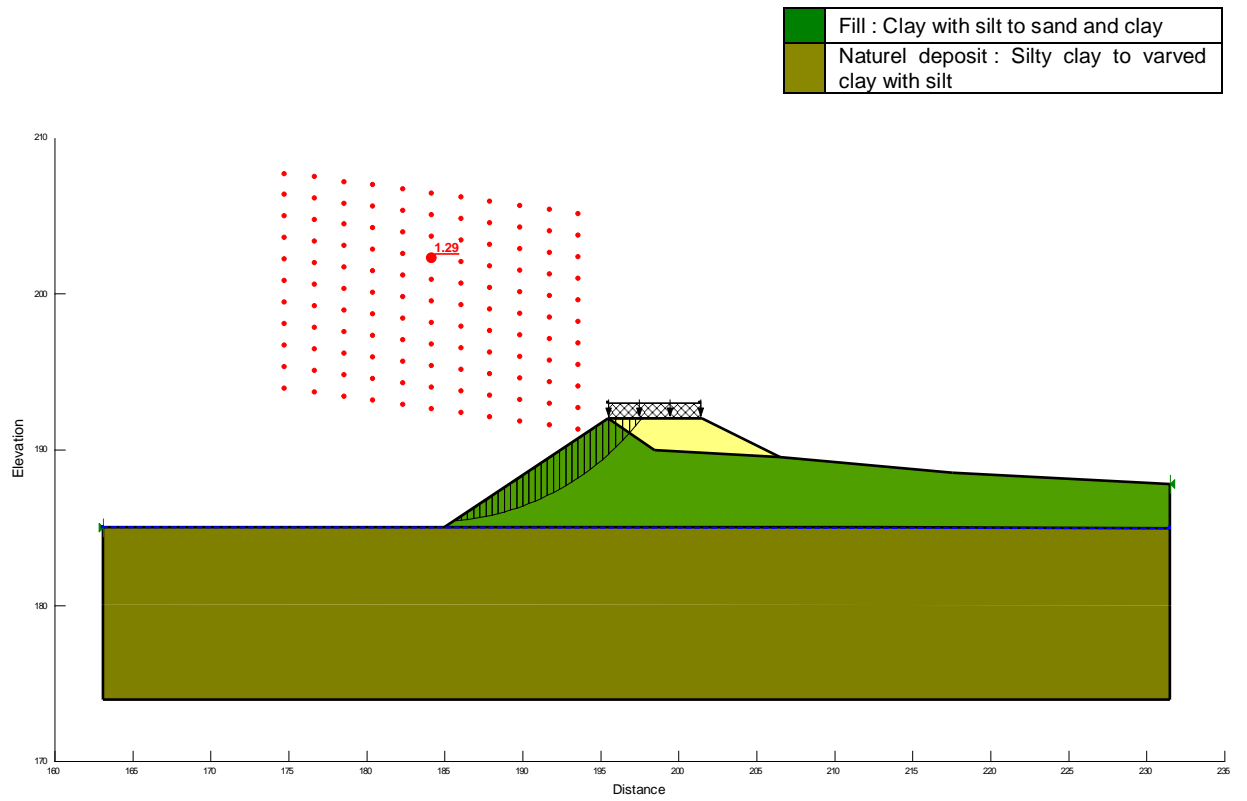


Figure 4: Long term stability analyses - Stage 1– Perpendicular to the road center line

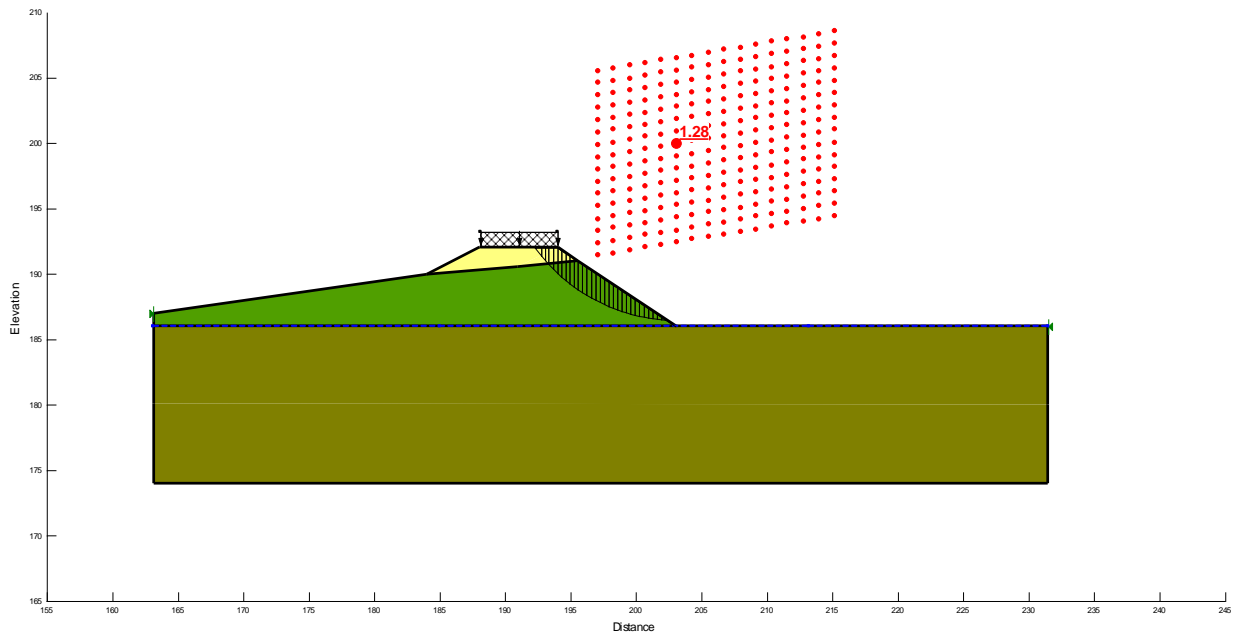


Figure 5: Short term stability analyses - Stage 2 – Perpendicular to the road center line

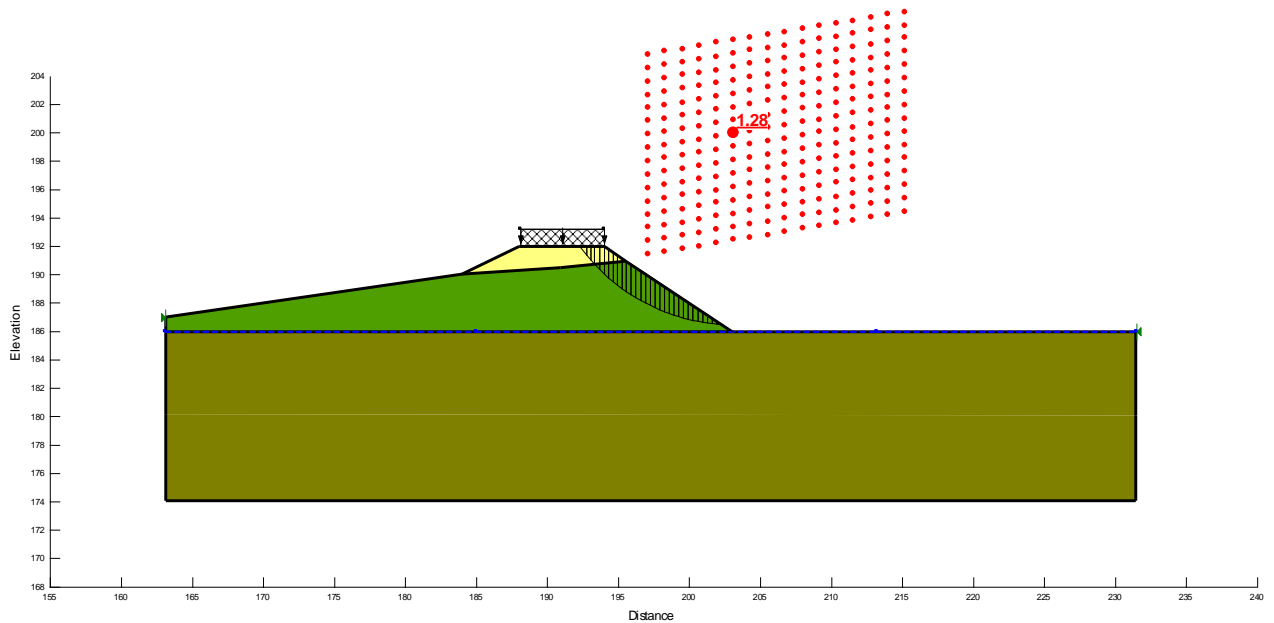


Figure 6: Long term stability analyses - Stage 2 – Perpendicular to the road center line

Results of the stability analyses indicate that the short and long term factors of safety obtained for the configuration of Stage 1 and Stage 2 are slightly below the required factor of safety (1.3). These results are considered to be marginally acceptable for the stability of these temporary slopes.

Temporary open cut excavations, perpendicular to centreline, can be carried out with the following restrictions. See Appendix 6 for Non Standard Special Provisions for the cut slopes at this culvert site.

- ▶ Temporary open cut less than or equal 7 m in height can be cut back to an angle of 1.5H:1V over the full height;
- ▶ Temporary open cuts of 5 m in height or greater cannot be left unattended unless a counterweight, consisting of a fill 1 m high by 3 m wide, is constructed along the full length of the toe of slope; this can be accomplished by bringing the pipe installation, backfilled to the obvert, up to the toe of the temporary slope;
- ▶ Temporary open cut greater than 7 m in height must be cut with an angle of 2H:1V over the whole height; This applies to the excavated transverse slopes when crossing the existing top of embankment;
- ▶ Temporary open cuts greater than 7 m in height must be inspected by a geotechnical engineer on a daily basis; Prior to commencing the excavation the geotechnical engineer must be given a minimum of 24 hours' notice;
- ▶ Excavations must be continuously 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. As such, the head of water adjacent to the excavation must be controlled during excavation and culvert installation. During construction, a cofferdam system

combined with 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. A sand bag or aqua dam type cofferdam, or possibly a temporary sheet piled cofferdam can be considered for controlling stream flow depending upon anticipated flow at time of construction. By-pass pumping through a separate diversion pipe, through the embankment, should be considered for diverting stream flow. Ultimately, the method of dewatering and stream 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 over-emphasized.

5.2.1.1 **Clay Rebound**

Replacing the culvert through open cut methods will involve temporarily removing up to about 7 m of fill from above the native clay deposits. This is equivalent to removing a load of about 110 kPa. Removing this confining pressure can potentially result in a rebound of the clay deposit. However, considering the limited time that the excavations will be open (i.e. one to two weeks) and the consolidation test results, it is estimated that rebound in the clay deposit will be in the order of 10 to 20 mm. When the embankment is reconstructed, assuming no net increase in loads, the clay deposit should be expected to settle an amount equal to the rebound.

5.2.2 **Temporary Earth Retaining Systems**

Several earth retaining systems are available for consideration for this project. For instance, a vertical wall installed along centreline for use as a temporary protection system can also be used to provide excavation support during the culvert replacement. The installation of a protection system for use in the culvert replacement operation will require penetration through some 7 m of embankment fill. The embankment fill is generally underlain by stiff to firm silty clays, which extended to the depth of borehole termination (elevation 178.5 m).

One possible method of constructing a temporary vertical wall for roadway protection along the centreline of the highway alignment would be to install (drive or vibrate) steel sheet piles through the embankment fill into the native material. It is noted that the previous investigation carried out in 1961 identified rock fill in the original, lower part of the embankment, however this current investigation did not encounter any apparent rock fill obstructions within the embankment fill. If a cobble/small boulder size obstruction 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 driving to the required depth. For reference purposes, borehole logs and a site plan from the 1961 investigation have been included in Appendix 5.

If a centreline protection system is considered, the side slopes (perpendicular to the culvert orientation) will have to be cut back on a 2H:1V since the cut height will be greater than 7 m. This will require an open excavation, a minimum 33 m wide at the road surface. It may be more cost effective to create a more narrow sheeted trench excavation, sufficiently wide to allow replacement and backfilling operations to be carried out.

As an indication, Table 4 below presents several shoring alternatives for comparison.

Table 4: Comparison of shoring alternatives

METHOD	DEPTH RANGE (M)	ADVANTAGES	DISADVANTAGES	REMARKS	ESTIMATED COSTS
Wood sheeting	1.5 – 5	<ul style="list-style-type: none"> • Low cost • Easily installed in good ground conditions 	<ul style="list-style-type: none"> • Limited by soil conditions • Limited depth of installation, • Low strength, • Discontinuous 	Not appropriate due to depth	\$ 650/m ²
Steel Sheet Piles	5 – 21	<ul style="list-style-type: none"> • High strength, continuous, • Readily available 	<ul style="list-style-type: none"> • Limited by soil conditions (i.e. obstructions) 	Recommended for temporary protection.	\$ 650/m ²
Soldier piles and timber lagging	5 – 25	<ul style="list-style-type: none"> • Easy installation • Readily available • Adaptable to various ground conditions 	<ul style="list-style-type: none"> • Pre-drilling may be required • Possible ground loss 	Not considered appropriate due to complexity, depth of excavation and cost	\$ 725/m ² Predrilling \$ 1,500/m ²
Micropiles with reinforced shotcrete face		<ul style="list-style-type: none"> • Can be installed in various ground conditions • High strength • Good tolerance 	<ul style="list-style-type: none"> • High Cost • Requires specialized equipment 	Not Considered due to higher costs	\$ 1,200 – 1,500/m ²

Design of a temporary earth retaining systems involves consideration of the stratigraphy of the subsurface material at the site under investigation, the position of the water table, and existing works nearby. Reference is given to OPSS 538 and OPSS 539 which pertain to excavation and support systems. The contractor's shoring/protection system design must be carried out by a qualified geotechnical engineer with appropriate experience.

The geotechnical parameters noted in the following Table 5 are suggested for the design of earth retaining systems.

Table 5: Geotechnical parameters to be used for the design of temporary earth retaining system

PARAMETER	GRANULAR EMBANKMENT	CLAY WITH SILT (FILL)	CLAY AND VARVED CALY DEPOSIT	
			SHORT TERM	LONG TERM
Unit weight , γ (kN/m ³)	20	17	17	
Submerged unit weight, γ' (kN/m ³)	10	7	7	
Effective cohesion, c' (kPa)	---	7	---	7
Effective friction angle ϕ' (°)	30	30	0	30
Undrained shear strength, S_u (kPa)	---	---	See Section 4	---
Coefficient of Active earth pressure (K_a)*	0.33	0.33	---	0.33
Coefficient of earth pressure at rest (K_o)*	0.5	0.50	---	0.50
Coefficient of passive earth pressure (K_p)*	3.00	3.00	---	3.00
Anchorage Coefficient (K_t)	1.2	---	---	---
Reduction Factor (α_c)	---	0.45	If $S_u > 100$ – 0.45 If $S_u < 100$ – 0.70	---
* For a vertical wall and a supported horizontal soil surface.				

Surcharge loads must be included in the lateral pressure calculations. Due to the height of the potential excavation, a tieback or earth anchor system will likely be required.

For support systems in the cohesionless embankment fills, a rectangular apparent pressure distribution over the height of the cut would be appropriate for design of the temporary shoring. The width of the apparent rectangular pressure distribution, over the height of excavation, can be considered equal to $0.65 \cdot K_a \cdot \gamma \cdot H$, where:

K_a = active earth pressure,

γ = unit weight, and

H = height of wall above the base of excavation.

For tieback systems in cohesive materials (i.e. clays, clayey fills), the pressure distribution is dependent on the shear strength of the clay and the results of total stress analyses in the material. To calculate the apparent pressure in soft to firm cohesive deposits, the equation $\gamma \cdot H - m \cdot 4 \cdot S_u$ can be used, where:

γ = unit weight,

H = height of wall above the base of excavation.

$m = 0.4$ if the soft/firm deposit extends below the base of excavation or

$= 1.0$ if a more resistant layer is present at or near the base of excavation

S_u = average undrained shear strength

To calculate the apparent pressure in stiff to hard cohesive deposits, the appropriate calculation is dependent on the total stress analysis, and generally ranges from $0.2 \cdot \gamma \cdot H$ to $0.4 \cdot \gamma \cdot H$, where:

γ = unit weight,

H = height of wall above the base of excavation.

For a tieback/earth anchor systems, the pull-out resistance (R) for tremie-grouted anchors in cohesionless soils can be estimated from the following equation as given in the Canadian Foundation Engineering Manual (4rd Edition):

$$R = \sigma_z' \cdot A_s \cdot L_s \cdot K_f \quad \text{Where:}$$

σ_z' = effective vertical stress at the midpoint of the load carrying length

A_s = effective unit surface area of the anchor

L_s = effective embedment length of the anchor

K_f = anchorage coefficient dependent on the soil type and conditions as provided in Table 5.

The pull-out resistance (R) for grouted anchors in clay soils can be estimated from the following equation:

$$R = \alpha_c \cdot A_s \cdot L_s \cdot S_u \quad \text{Where:}$$

α_c = reduction factor as provided in Table 5,

A_s = effective unit surface area of the anchor bond zone

L_s = effective length of anchor bond zone

S_u = average undrained shear strength of clay

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 (4rd 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 subject to the design and installation being in accordance with the anchor system manufacturer's requirements.

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

5.2.3 Trenchless/Tunnelling Techniques

The two boreholes advanced adjacent to the existing culvert (Borehole Nos. 2 and 3) indicate that the embankment/embedment fill consists of some 1.8 to 2.1 m of sand fill overlying 1.7 to 3.0 m of silty clay fill, followed by a sand and clay mix fill which extends to native subgrade at a depths of some 7.3 m. The water level, measured at Borehole Nos. 2 and 3, at elevations 187.9 and 188.3 m, respectively,

was somewhat elevated above the existing culvert invert. The potential for the presence of saturated sand seams/pockets to be encountered will control the method of trenchless technology considered. This water level is also expected to vary seasonally/yearly. It must also be noted that the stratigraphy between boreholes may vary and areas of softer or more dense soils may be present along the proposed culvert alignment.

The replacement culvert currently under consideration is a 2700 mm diameter smooth wall concrete pipe. It is understood that consideration has also been given to lining the existing culvert. If the existing 2.3 m SPCSP pipe is lined, it is understood that a 1200 mm diameter would have to be used as a liner since the existing pipe has sagged towards the middle, and there is a deformation in the upper part of the existing culvert at about 1/3 distance (approximately 20 m) from the end. This would require installation of a second pipe of 1800 mm in diameter adjacent to the lined existing 2300 mm culvert to provide sufficient capacity to accommodate the 25 Year Storm and greater flood waters.

The four foundation boreholes advanced through the embankment for this investigation did not encounter cobble or boulder obstructions. However, in the eight boreholes which were advanced to a 2.6 m depth for the pavement design, occasional cobbles were noted in the embankment fill. As such there is a risk that cobble/boulder size obstructions may be encountered during the boring operation and this condition will impact the type of method chosen by the tunneling contractor. The geotechnical data for borings at Station 11+802 to 11+836 Dymond Township which were advanced during the geotechnical investigation are included in Appendix 6.

5.2.3.1 ***Horizontal Directional Drilling***

In a Horizontal Directional Drilling (HDD) operation a boring is advanced horizontally through the embankment along the proposed alignment uncased, and drilling fluids are used to keep the boring open. Generally, an initial small diameter bore is advanced and then a reamer is pulled through to open up the hole to the required size and the pipe/casing pulled through. A HDD operation would have to be carried out in accordance with OPSS 450 – Construction Specification for Pipeline and Utility Installation in Soil by Horizontal Directional Drilling.

The required diameter of a 2700 mm diameter replacement pipe or the new 1800 mm diameter secondary pipe would eliminate the use of Horizontal Directional Drill (HDD) since this method is limited to pipes having a diameter of 1200 mm or less. As such, the remaining three trenchless technologies will be reviewed. In addition, since the bore is advanced uncased there is the risk of loss of ground which could result in sink holes developing above the route alignment. As such, this method is not considered to be appropriate for this project and will not be discussed further.

5.2.3.2 ***Jack and Bore***

A jack and bore method typically involves jacking a pipe and internal auger section along the alignment from a jacking pit. Depending on the subsurface and groundwater conditions, the auger is generally advanced beyond the face of the casing to remove the soil and allow jacking of the casing/pipe. Since the embankment contains pockets/layers of saturated sand, there is an elevated risk of ground subsidence due to potential running of wet soil entering the boring. To eliminate this risk, groundwater control measures would be required along the bore. An unwatering system, such as well points, would

be additional cost and difficult to install below the existing travelled road surface and may not intersect an isolated saturated sand pocket(s) which could lead to a running sand condition at the advancing face. Jack and Bore installation should be carried out in accordance with OPSS 416, Construction Specifications for Pipeline and Utility Installation by Jacking and Boring. Jack and bore operations are generally limited to pipes with a diameter of less than 1500 mm diameter.

Because of the risk of ground loss due to the possible presence of saturated sand layers/seams and obstructions such as cobbles/boulders, this method is not considered appropriate at this site.

5.2.3.3 *Pipe Jacking/Micro-Tunneling*

Pipe jacking/micro-tunneling operations are carried out in a similar manner as a jack and bore operation, however, to control flowing wet soil and changing/mixed ground conditions the micro-tunneling machine/shield is advanced with a closed face which allows controlled removal of cuttings/spoil. The pipe sections (concrete, steel, or fiberglass) are jacked forward, from the jacking pit, with the shield attached to the lead pipe. The cuttings/spoils are removed on a dolly or conveyor system as the shield advances. Different shields are available for varying ground conditions, from picks/teeth for soft ground to disc cutters with cone crushers to accommodate broken rock/cobble boulder pieces/mixed ground.

A micro-tunneling machine can fit into a 3 m diameter manhole, to allow working in congested urban areas. However, at this site, the Contractor will likely excavate into the embankment, along the pipe alignment, until there is a face of some 4 to 5 m height, thereby reducing the length of bore, the risk associated with micro-tunneling methods, and subsequently the owners cost. The Contractor would then install a trench box of sufficient width (generally a minimum 5 m wide for installation of a 2.7 m diameter pipe) and length to potentially allow for sections of pipe, jacking equipment and removal of spoils. The trench box would also allow installation of a back plate to act as a thrust block.

If the existing culvert is lined, micro-tunneling could be considered to advance a tunnel for the 1800 mm diameter secondary pipe parallel to the existing culvert to install a new culvert to increase capacity as previously discussed. Alternatively, micro-tunneling could be used to install a new 2700 mm concrete pipe culvert. Either sized culvert would have to be off-set a **minimum** of one pipe diameter or 2 m from the existing alignment to minimize the risk of penetrating the existing embedment material or possibly intercepting the existing SPCSP. Since the length of bore would be reduced by excavating into the existing embankment for a trench box, the offset would have to be sufficient to allow for box installation. Under these current conditions, a minimum offset of 3 m should be considered.

It may be necessary to inject a bentonite/polymer grout along the exterior of the pipe for lubrication to decrease the required thrust. It should be noted that the depth of embankment above the pipe is only about 4 m and, as such, the Contractor will have to maintain strict control of the grout/face pressure and grout take to ensure that the pressures associated with tunneling does not lift the surface of the road.

5.2.3.4 *Pipe Ramming*

Horizontal pipe ramming involves driving an open ended steel casing using from a percussive hammer. During the driving operation the soil being penetrated fills the casing. This allows the casing to be

advanced with minimal groundwater control since the soil plugs the casing and is not removed until the casing enters the receiving pit or breaks through the opposite embankment slope. The soil is then generally removed with an auger; however soil removal can also be completed using controlled water pressure and a vacuum system. Pipe ramming can be advanced through soils containing cobbles and boulders although the degree of difficulty would increase and obstructions have to be smaller than the casing diameter.

5.2.3.5 *Site Specific Recommendations*

Table 6 below identifies the trenchless technologies that were considered for this project and the comparable advantages and disadvantages of each method.

Table 6: Comparison of trenchless techniques

METHOD	ADVANTAGES	DISADVANTAGES
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 shafts • Groundwater control is required for the bore and shafts • Elevated potential for ground subsidence • Boring diameter 1 m plus required to allow removal of cobbles/boulders • Diameter range generally 200 to 1500 mm
Pipe Ramming	<ul style="list-style-type: none"> • Minimal groundwater control required along the installation route (unless required to remove obstruction/old pipe) • Can penetrate soils containing cobbles/boulders if obstruction less than casing diameter. 	<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 • 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
Horizontal Direction Drilling	<ul style="list-style-type: none"> • Can be used in most ground condition • Generally does not require staging pits 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

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.

Both Pipe Ramming and Micro-Tunneling are technically feasible at this site, however, availability of specific equipment (rings, hammers, etc.) required to pipe ram the different diameters may limit interest from the limited number of available specialist contractors capable of carrying out these operations. However, it is suggested that micro-tunneling could be considered for this crossing/installation.

5.2.3.6 **Monitoring**

The ground surface, below the tunneling route, may become distorted (i.e. settlement/ground loss or heave due to system pressure). The more frequent condition likely to develop is surface settlement due to ground loss at the advancing face. Heave of the embankment surface can also result from uncontrolled face or grouting pressure and drilling fluids (bentonite) may return to the surface (frack out) depending upon installation type. This is particularly a concern where there is potential for the drilling fluids to frack out into the aquatic environment (creek/river). These issues are generally prevented by good construction practices and detailed planning and monitoring.

Prior to commencement of the selected operation, the Contractor must provide a detailed plan outlining his methodology with stamped drawings identifying load/pressure calculations. A monitoring program with surface monitoring points must also be established at each tunneling site and implemented in accordance with the approved monitoring plan. Guidelines for settlement monitoring are included in Appendix 6.

5.2.4 **Cost Estimates**

Cost estimates for culvert replacement have been developed by LVM | Merlex for the Ministry's consideration to compare the above discussed methods of culvert installation. There are two culverts in deep fills that have been considered in this assignment. These culverts are at Stations 11+814 and 12+548 Dymond Township. The latter culvert is discussed in a separate Foundation Investigation and Design Report (Geocres No. 31M-103). Since both of these culverts will be replaced during the same contract, the same options for both culverts have also been included in the total cost estimates for the same options, in the summary table, for information purposes.

The options are as follows:

- Option A – Carry out open excavation with staging as discussed in Section 5.2.1

- ▶ Option B-I – Install temporary support along the length of the highway centerline and carry out open excavation for culvert installation
- ▶ Option B-II Install a short section of temporary along centerline, say 5 m long, and then develop a sheeted excavation perpendicular to centerline, for temporary support during culvert installation. The sheeted trench would extend to a point where the depth of excavation is less than 4 m, at which point an open cut excavation would be more economical.
- ▶ Option C – Install a 1200 mm diameter liner in the existing 2300 mm diameter culvert. To handle periods of higher flow a second culvert would be installed by Micro-tunneling. It is understood that based on hydrology requirements, this secondary culvert would have to be 1800 mm in diameter.
- ▶ Option D – Install a new 2700 mm diameter culvert by pipe jacking/micro-tunneling. Considering the relatively flat slopes at this site, it is likely the tunneling contractor will limit the length of his bore and install the inlet and outlet by open cut methods. This assumption has been considered in the cost estimating.

The following Table 7 and Figure 7 summarizes the cost estimates for the culvert at Station 11+814 Dymond, as well as a Total Cost Estimate to complete the culvert replacements at both Station 11+814 and Station 12+548 under the same contract.

Table 7: Summary of culvert replacement cost estimate

OPTION	COST ESTIMATE (\$)	
	STATION 11+814	STATIONS 11+814 AND 12+548
A	677,501.00	1,422,980.00
B-1	601,130.00	1,485,630.00
B-2	732,830.00	1,726,370.00
C	490,902.00	1,211,517.00
D	660,870.00	1,476,285.00

SUMMARY OF CULVERT REPLACEMENT COST ESTIMATE

GWP 5574-04-00, Highway 65
Station 11+814
Dymond Township

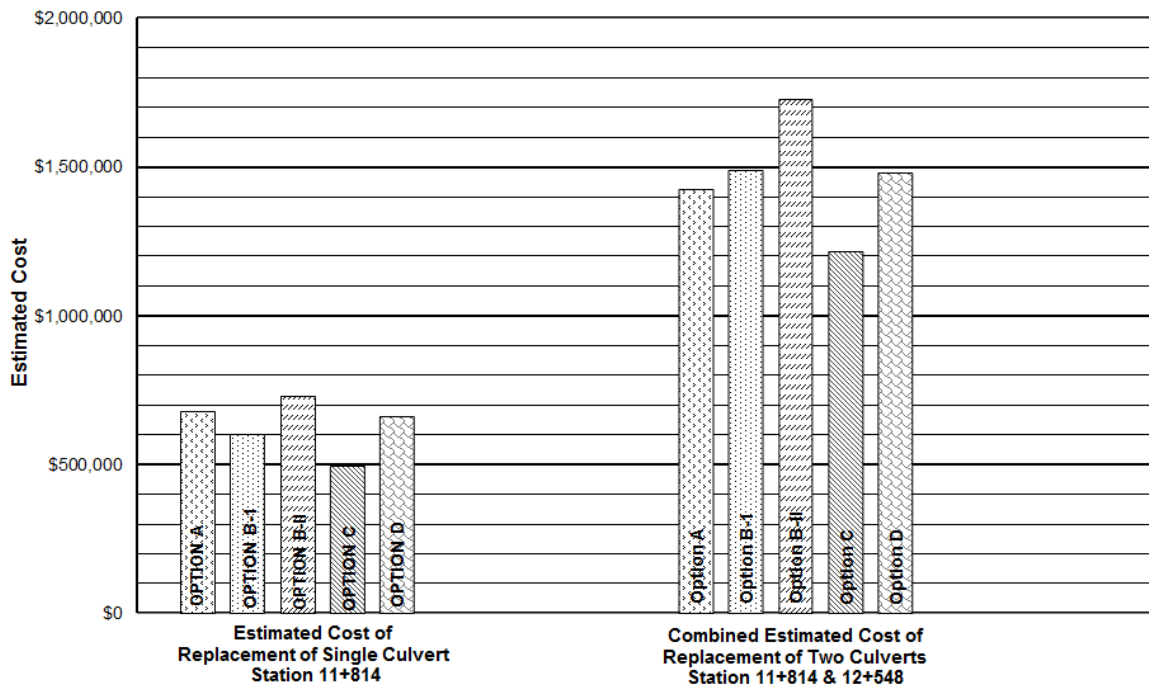


Figure 7: Summary of culvert replacement cost estimate

Based on the above comparison, Option C appears to be the most cost effective method of installing the culvert(s).

5.3 FOUNDATION CONSIDERATIONS

The founding native stiff to firm silty clays present below the existing embankment have undergone consolidation settlement, plus strength gain, due to the dead load of the existing embankment, which has been in-place since the embankment was reconstruction in the mid 1960's. At that time it was estimated that the embankment would settle some 910 mm (36 inches), due to the increase in vertical effective stress associated with the implemented 2.4 m grade raise. The report also indicated that this settlement would occur rapidly, estimated at some 6 months. As such the flexible culvert was placed on a 450 mm (18 inch) camber. Based on a visual review of the internal culvert alignment, (see Photo Essay, Appendix 4), these settlement estimates appear to have been reasonable.

Based on the characteristics of the native silty clay subgrade soils, present below the culvert, the response of the existing embankment and a founding elevation similar to that of the existing culvert, a factored geotechnical resistance at ULS of 160 kPa can be used for design of a closed culvert (i.e. precast concrete frame box culvert, SPCSPA or SPCSP culvert).

The geotechnical reaction at Serviceability Limit State is the soil reaction at a specific deformation level. As noted above the underlying soils are considered normally consolidated as such any appreciable increase in sustained (dead) load will result in consolidation settlement of the underlying clay deposit. In consideration of 25 mm settlement of the culvert the Geotechnical Reaction of 100 kPa applies for this serviceability level. If this Geotechnical Reaction value is exceeded, larger settlements will develop. This would be the case if the culvert is lined, with a 1.2 m diameter liner, (and normal 40 MPa grout used to fill the annular void) or if the full 2.3 m diameter culvert is abandon and fully grouted. If the culvert is lined the geotechnical reaction would be in the order of 120 kPa or if the culvert is abandon and fully grouted the geotechnical reaction would be in the order of 135 kPa. We have estimated that the resulting deformations/settlements, at the above noted geotechnical reactions, would be in the order of 75 and 100 mm, respectively. The magnitude of settlement could be reduced if a lightweight product, such as a cellular concrete, were used to fill the culvert voids.

The above settlement considerations are for stress increases directly below the culvert element(s) at the elevation of the existing culvert. If an area load increase associated with a grade raise is considered, this stress increase will extend to a substantially greater depth and could result in larger embankment settlements. As noted previously, a grade raise is not presently planned at this location. If during detailed design a grade raise is considered, further geotechnical review will be required.

5.4 TRENCH BACKFILLING

If an open cut excavation, with or without shoring, is carried out, the concrete pipe culvert will have to be properly bedded, covered, and backfilled.

A Class B Bedding for the concrete pipe 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 300 mm. Cover material for concrete pipe can consist of Granular A and placed to the dimensions as shown on OPSD 802.031.

The bedding below the concrete pipe should be shaped and lightly compacted to accommodate the curvature of the pipe, however the backfill under the haunches and to the sides of the pipe should be well compacted to 100% of Standard Proctor Dry Density (SPDD) in accordance with OPSS 501. The backfill should be placed and compacted simultaneously and to the same levels on either side of the pipe.

The native silty clays that will be excavated in an open cut operation can be used for backfill, up to the depth of frost penetration, provided it is at a moisture content that will allow effective compaction (within $\pm 2\%$ of its Standard Proctor Optimum Moisture Content). A bulk sample of the upper brown silty clay was obtained and a Standard Proctor Dry Density test was undertaken which revealed a SPDD of 1478 kg/m^3 at an optimum moisture content of 26.2% (see Figure P-1, Appendix 6). In order to effectively compact clay soils, the moisture content should be within 1 to 2% of the optimum value. As can be seen from the natural moisture content test results, as shown on the Record of Borehole logs, only the upper desiccated clay deposit has natural moisture contents in the range of $26\pm 2\%$ (i.e. 24 to 28%). As such, it is anticipated that the lower grey silty clay would have too high a natural moisture content for

effective compaction and should not be used for trench backfill. The excessively wet clay deposit could be reworked, to reduce its moisture content, however this involves spreading the material in thin lifts, some 200 to 300 mm thick and scarifying or discing the layer to allow sun and wind to evaporate the moisture. This can be a time consuming operation and is totally dependent of having continuously favorable weather (warm and dry) during the drying operations. Due to the extended time required to dry the soil and the risk of having unsuitable weather conditions (favorable weather generally limited to summer months) we do not recommend that this operation be considered.

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 a minimum 3 m in width, 500 mm thick and extend across the stream bed. Clay seals are generally used where significant head differences exist between the inlet and outlet of a culvert to prevent flow through the embankment. Considering the size and cohesive nature of the native soils, it is recommended that clay seals be installed at both the inlet and outlet ends of the culvert. At the inlet substantial grading may be carried out to align the stream. As such, the clay seal will have to extend transverse to the culvert, for the full width of the excavated face. Clay seals should be constructed as per OPSS 1205.

The embankment will be reconstructed to the same grade and width as the existing embankment. Since there will be no load increase on the underlying silty clay deposit, no appreciable settlement of the new culvert is anticipated and as such, it will not be necessary to install the new culvert or secondary culvert (if necessary) on a camber.

6 STATEMENT OF LIMITATIONS

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

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

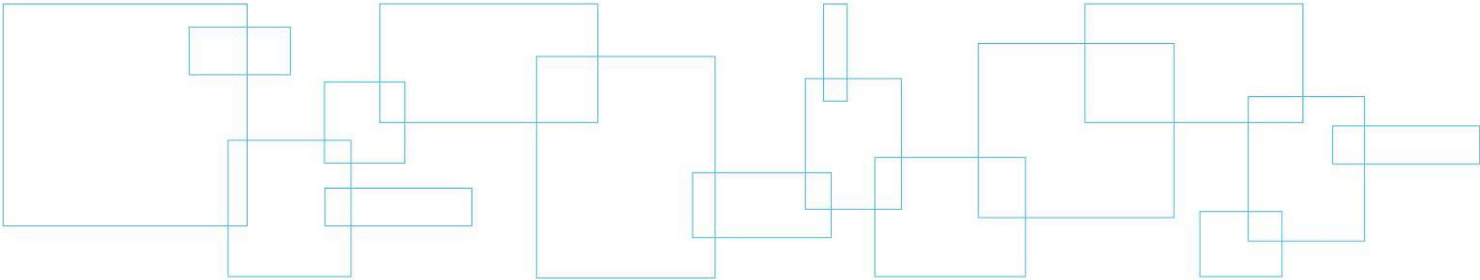
Proper subgrade preparation, groundwater control, compaction, etc. are all critical aspects of the bearing capacity of native soils. It must be noted that different aspects of the geotechnical design are based on the assumption that LVM | MERLEX will be retained during site preparation and construction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

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

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

Appendix 1 Key Plan

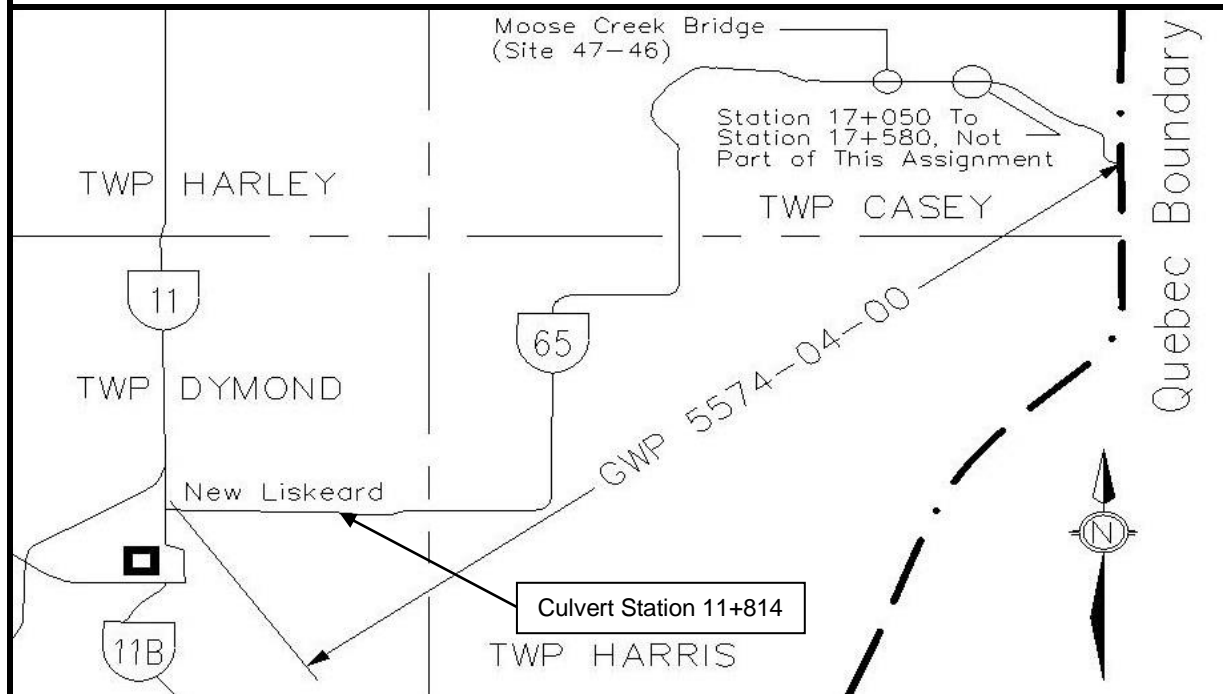
Drawing No. 1 Key Plan



KEY PLAN

Drawing No. 1

NOT TO SCALE



**FINAL FOUNDATION
INVESTIGATION AND DESIGN REPORT
GWP 5574-04-00**

Highway 65

From 0.1 km East of Armstrong Street
Easterly 22.5 km To the
Ontario/Québec Boundary

MEL Ref. No.: 12/08/12028-F1

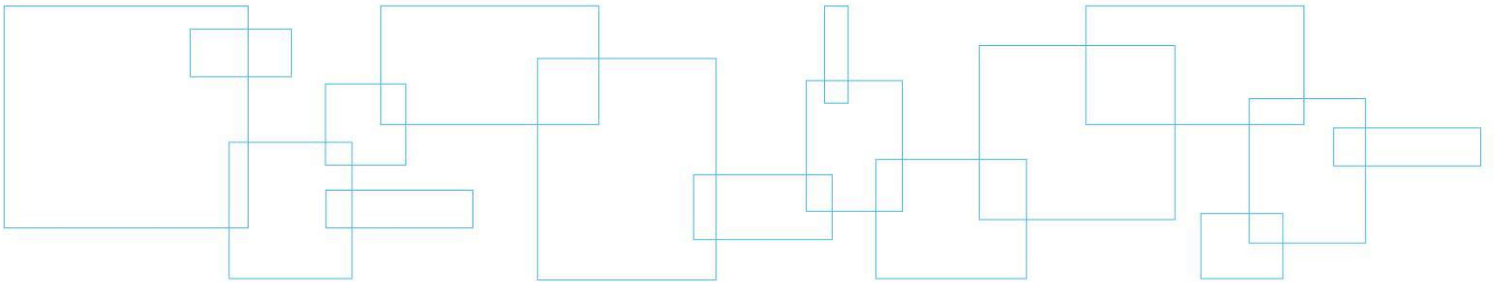
December 2013

LVM | MERLEX

Appendix 2 Subsurface Data

Enclosure No. 1
Enclosure Nos. 2 to 7

List of Abbreviations and Symbols
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. 1



REFERENCE	12/03/12028-F1	DATUM	Geodetic	LOCATION	N 5265751.3 E 406421.8 - Dymond Township Station 11+800	ORIGINATED BY	JL
PROJECT	GWP 5574-04-00, Highway 65			BOREHOLE TYPE	Track Mounted CME 45B - Hollow Stem Augers	COMPILED BY	AT/RG
CLIENT	AECOM Inc.	DATE (Started)	5 November 2012	TIME		CHECKED BY	MAM
		DATE (Completed)	5 November 2012	(Completed)	2:30:00 PM		

[illegible]

MEL-GEO 12028 - AREA A - BOREHOL LOGS - FINAL.GPJ MEL-GEO.GDT 18/6/13

METRIC**RECORD OF BOREHOLE NO. 1**

REFERENCE 12/03/12028-F1 DATUM Geodetic LOCATION N 5265751.3 E 406421.8 - Dymond Township Station 11+800 ORIGINATED BY JL
 PROJECT GWP 5574-04-00, Highway 65 BOREHOLE TYPE Track Mounted CME 45B - Hollow Stem Augers COMPILED BY AT/RG
 CLIENT AECOM Inc. DATE (Started) 5 November 2012 TIME
 DATE (Completed) 5 November 2012 (Completed) 2:30:00 PM CHECKED BY MAM

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA (SI CL)
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE			"N" VALUES	20	40	60	80					
	Continued from Previous Page															
149.2 38.7	DCPT Refusal End of Borehole															

MEL-GEO 12028 - AREA A - BOREHOLE LOGS - FINAL.GPJ MEL-GEO.GDT 18/6/13



METRIC

RECORD OF BOREHOLE NO. 2



REFERENCE 12/03/12028-F1 DATUM Geodetic LOCATION N 5265720.3 E 406431.9 - Dymond Township Station 11+810 ORIGINATED BY JL
 PROJECT GWP 5574-04-00, Highway 65 BOREHOLE TYPE Truck Mounted CME 45B - Hollow Stem Augers COMPILED BY AT
 CLIENT AECOM Inc. DATE (Started) 15 November 2012 TIME 15 November 2012 (Completed) 1:15:00 PM 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	PLASTIC LIMIT W _p NATURAL MOISTURE CONTENT W LIQUID LIMIT W _L WATER CONTENT (%)	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	"N" VALUES						
193.0	Ground Surface										
0.0	± 75 mm Asphalt ± 150 mm Crushed Gravel FILL - brown sand some silt with gravel (loose/compact)		1	SS	25						
			2	SS	20						
191.2			3	SS	5						
1.8	FILL - grey silty clay trace asphalt 25 mm asphalt layer		4	SS	16						
			5	SS	14						
189.5			6	SS	15						
3.5	FILL - grey sand and clay with gravel some silt		7	SS	22						
			8	SS	20						
			9	SS	22						
			10	SS	26						
185.7			11	SS	12						
7.3	CLAY - grey clay with silt trace organics and wood to a ± 7.9 m depth		12	SS	WH						
			13	TO	PM						
182.3											
10.7	CLAY - grey clay with silt varved structure (±25 mm thick clay layers with ±6 mm thick silty clay varves) (firm/stiff) Continued Next Page										
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 1) 15/11/12 12:43:00 PM 2) 3)			
								Water Depth (m) 5.1 - -			
								Cave In (m) 12.9 - -			

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



METRIC**RECORD OF BOREHOLE NO. 2**

REFERENCE 12/03/12028-F1 DATUM Geodetic LOCATION N 5265720.3 E 406431.9 - Dymond Township Station 11+810 ORIGINATED BY JL
 PROJECT GWP 5574-04-00, Highway 65 BOREHOLE TYPE Truck Mounted CME 45B - Hollow Stem Augers COMPILED BY AT
 CLIENT AECOM Inc. DATE (Started) 15 November 2012 TIME
 DATE (Completed) 15 November 2012 (Completed) 1:15:00 PM CHECKED BY MAM

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT		UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA (SI CL)	
ELEV DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa		W _p W W _L				
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE	WATER CONTENT (%)					
	Continued from Previous Page							20 40 60 80 100		20 40 60				
	CLAY - grey clay with silt varved structure (±25 mm thick clay layers with ±6 mm thick silty clay varves) (firm/stiff)		14	SS	PM	單	180							
			15	TO	PM		179							
178.5														
14.5	End of Sampling End of Borehole													

MEL-GEO 12028 - AREA A - BOREHOLE LOGS - FINAL.GPJ MEL-GEO.GDT 18/6/13



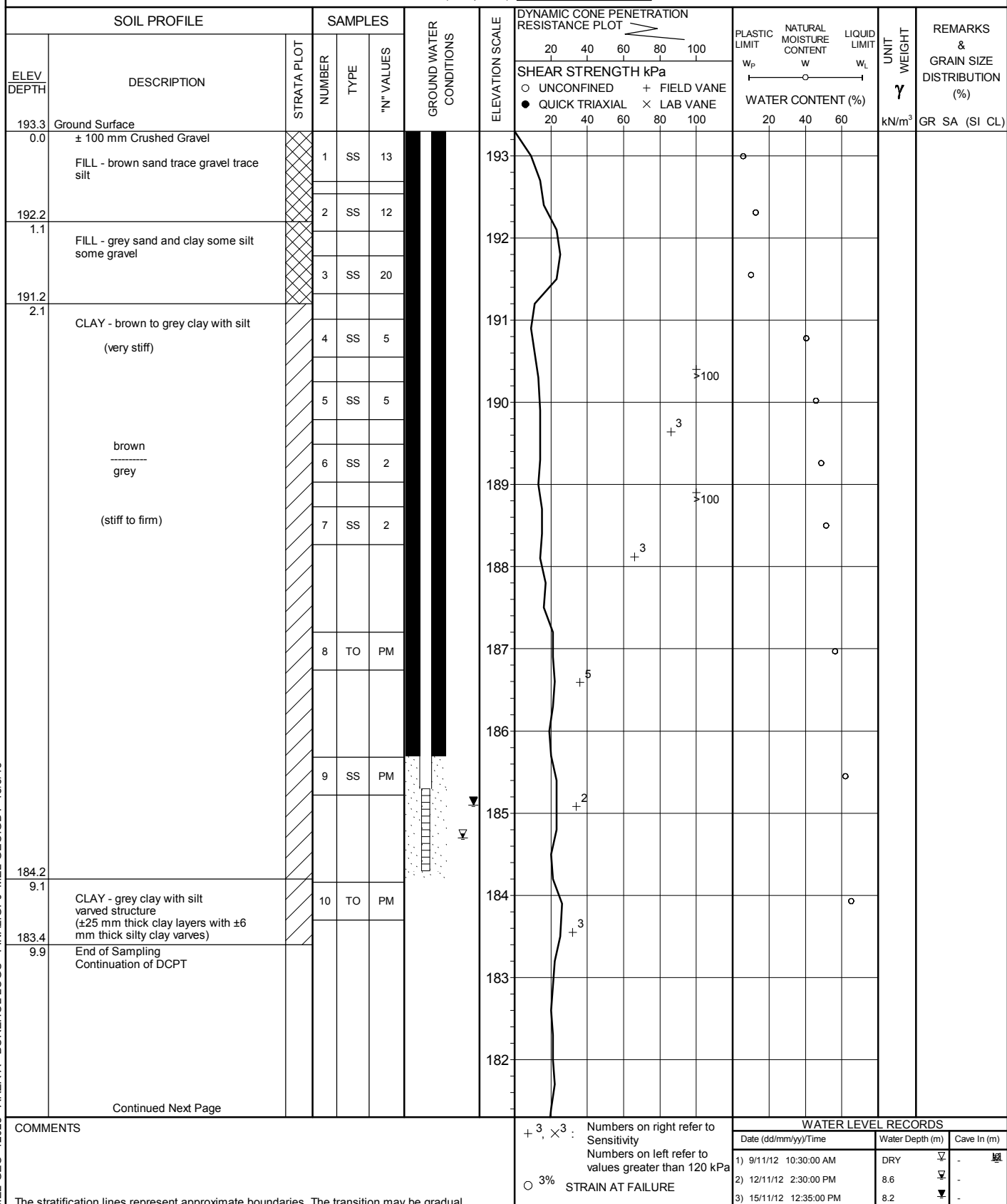
METRIC**RECORD OF BOREHOLE NO. 3**

REFERENCE 12/03/12028-F1 DATUM Geodetic LOCATION N 5265712.5 E 406439.0 - Dymond Township Station 11+817 ORIGINATED BY JL
 PROJECT GWP 5574-04-00, Highway 65 BOREHOLE TYPE Truck Mounted CME 45B - Hollow Stem Augers COMPILED BY AT
 CLIENT AECOM Inc. DATE (Started) 15 November 2012 TIME
 DATE (Completed) 16 November 2012 (Completed) 9:40:00 AM CHECKED BY MAM

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	"N" VALUES			20	40					
	Continued from Previous Page													
	CLAY - grey clay with silt varved structure		13	TO	PM									
	(±25 mm thick clay layers with ±6 mm thick silty clay varves)													
179.8	(firm)													
13.0	End of Sampling End of Borehole													

MEL-GEO 12028 - AREA A - BOREHOLE LOGS - FINAL.GPJ MEL-GEO.GDT 18/6/13





METRIC

RECORD OF BOREHOLE NO. 4



REFERENCE	12/03/12028-F1	DATUM	Geodetic	LOCATION	N 5265712.0 E 406415.9 - Dymond Township Station 11+784	ORIGINATED BY	JL
PROJECT	GWP 5574-04-00, Highway 65			BOREHOLE TYPE	Truck Mounted CME 45B - Hollow Stem Augers	COMPILED BY	AT
CLIENT	AECOM Inc.			DATE (Started)	9 November 2012	TIME (Completed)	11:00:00 AM
				DATE (Completed)		CHECKED BY	MAM

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL LIMIT MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA (SI CL)
ELEV DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	"N" VALUES		SHEAR STRENGTH kPa			WATER CONTENT (%)				
							○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE							
	Continued from Previous Page						20 40 60 80 100	20 40 60	W _p W W _L					
							181							
							180							
							179							
							178							
							177							
							176							
							175							
							174							
							173							
							172							
							171							
							170							
							169							

MEL-GEO 12028 - AREA A - BOREHOL LOGS - FINAL.GPJ MEL-GEO.GDT 18/6/13

METRIC**RECORD OF BOREHOLE NO. 4**

REFERENCE 12/03/12028-F1 DATUM Geodetic LOCATION N 5265712.0 E 406415.9 - Dymond Township Station 11+784 ORIGINATED BY JL
 PROJECT GWP 5574-04-00, Highway 65 BOREHOLE TYPE Truck Mounted CME 45B - Hollow Stem Augers COMPILED BY AT
 CLIENT AECOM Inc. DATE (Started) 9 November 2012 TIME (Completed) 11:00:00 AM CHECKED BY MAM
 DATE (Completed)

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					
	Continued from Previous Page						○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE						
						168							
						167							
						166							
						165							
						164							
						163							
						162							
						161							
						160							
						159							
						158							
						157							
						156							
	Continued Next Page												

MEL-GEO 12028 - AREA A - BOREHOLE LOGS - FINAL.GPJ MEL-GEO.GDT 18/6/13



METRIC

RECORD OF BOREHOLE NO. 4



REFERENCE 12/03/12028-F1 DATUM Geodetic LOCATION N 5265712.0 E 406415.9 - Dymond Township Station 11+784 ORIGINATED BY JL
 PROJECT GWP 5574-04-00, Highway 65 BOREHOLE TYPE Truck Mounted CME 45B - Hollow Stem Augers COMPILED BY AT
 CLIENT AECOM Inc. DATE (Started) 9 November 2012 TIME (Completed) 11:00:00 AM CHECKED BY MAM
 DATE (Completed)

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE			"N" VALUES	20					
	Continued from Previous Page												
153.8						155							
39.5	DCPT Refusal End of Borehole					154							

MEL-GEO 12028 - AREA A - BOREHOLE LOGS - FINAL.GPJ MEL-GEO.GDT 18/6/13



METRIC

RECORD OF BOREHOLE NO. 5



REFERENCE 12/03/12028-F1 DATUM Geodetic LOCATION N 5265722.3 E 406466.9 - Dymond Township Station 11+845 ORIGINATED BY JL
 PROJECT GWP 5574-04-00, Highway 65 BOREHOLE TYPE Truck Mounted CME 45B - Hollow Stem Augers COMPILED BY AT
 CLIENT AECOM Inc. DATE (Started) 12 November 2012 TIME (Completed) 2:00:00 PM CHECKED BY MAM
 DATE (Completed) 12 November 2012

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					
192.8 0.0	Ground Surface ± 100 mm Crushed Gravel		1	SS	16					
	FILL - brown sand trace gravel trace silt		2	SS	12					4 89 (7)
191.4 1.4	CLAY - brown clay some to with silt trace gravel		3	SS	12					
	(very stiff/firm)		4	SS	7					
			5	SS	6					0 0 12 88
	brown		6	SS	4					
	grey		7	SS	3					0 0 15 85
			8	TO	PM					
185.2 7.6	CLAY - grey clay with silt varved structure		9	SS	PM					0 0 18 82
	(±25 mm thick clay layers with ±6 mm thick silty clay varves)									
	(stiff to firm)		10	TO	PM					
			11	SS	WH					
	Continued Next Page									

COMMENTS
Borehole extended from 9.9 m to 12.9 m, June 7, 2013.

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) 12/11/12 1:50:00 PM	DRY	-
2) 13/11/12 3:15:00 PM	8.9	-
3) 15/11/12 12:35:00 PM	8.3	-

MEL-GEO 12028 - AREA A - BOREHOLE LOGS - FINAL.GPJ MEL-GEO.GDT 18/6/13

METRIC

RECORD OF BOREHOLE NO. 5



REFERENCE 12/03/12028-F1 DATUM Geodetic LOCATION N 5265722.3 E 406466.9 - Dymond Township Station 11+845 ORIGINATED BY JL
 PROJECT GWP 5574-04-00, Highway 65 BOREHOLE TYPE Truck Mounted CME 45B - Hollow Stem Augers COMPILED BY AT
 CLIENT AECOM Inc. DATE (Started) 12 November 2012 TIME
 DATE (Completed) 12 November 2012 (Completed) 2:00:00 PM CHECKED BY MAM

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE			"N" VALUES	20					
	Continued from Previous Page		12	SS	PM								
179.9						180							
12.9	End of Sampling End of Borehole												

MEL-GEO 12028 - AREA A - BOREHOLE LOGS - FINAL.GPJ MEL-GEO.GDT 18/6/13



METRIC

RECORD OF BOREHOLE NO. 6



REFERENCE 12/03/12028-F1 DATUM Geodetic LOCATION N 5265686.8 E 406444.0 - Dymond Township Station 11+822 ORIGINATED BY JL
 PROJECT GWP 5574-04-00, Highway 65 BOREHOLE TYPE Track Mounted CME 45B - Hollow Stem Augers COMPILED BY AT
 CLIENT AECOM Inc. DATE (Started) 6 November 2012 TIME
 DATE (Completed) 6 November 2012 (Completed) 11:30:00 AM 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				
187.6	Ground Surface		1	SS	4				
0.0	FILL - brown clay with silt occasional cobbles/boulders								
186.8			2	SS	4				
0.8	CLAY - grey clay some to with silt trace organics to a ± 2.7 m depth		3	SS	4				
			4	SS	4				
184.6			5	SS	WH				
3.0	CLAY - grey clay with silt varved structure (±25 mm thick clay layers with ±6 mm thick silty clay varves) (firm)		6	SS	PM				
			7	SS	PM				
			8	TO	PM				
180.7	End of Sampling End of Borehole								
6.9									

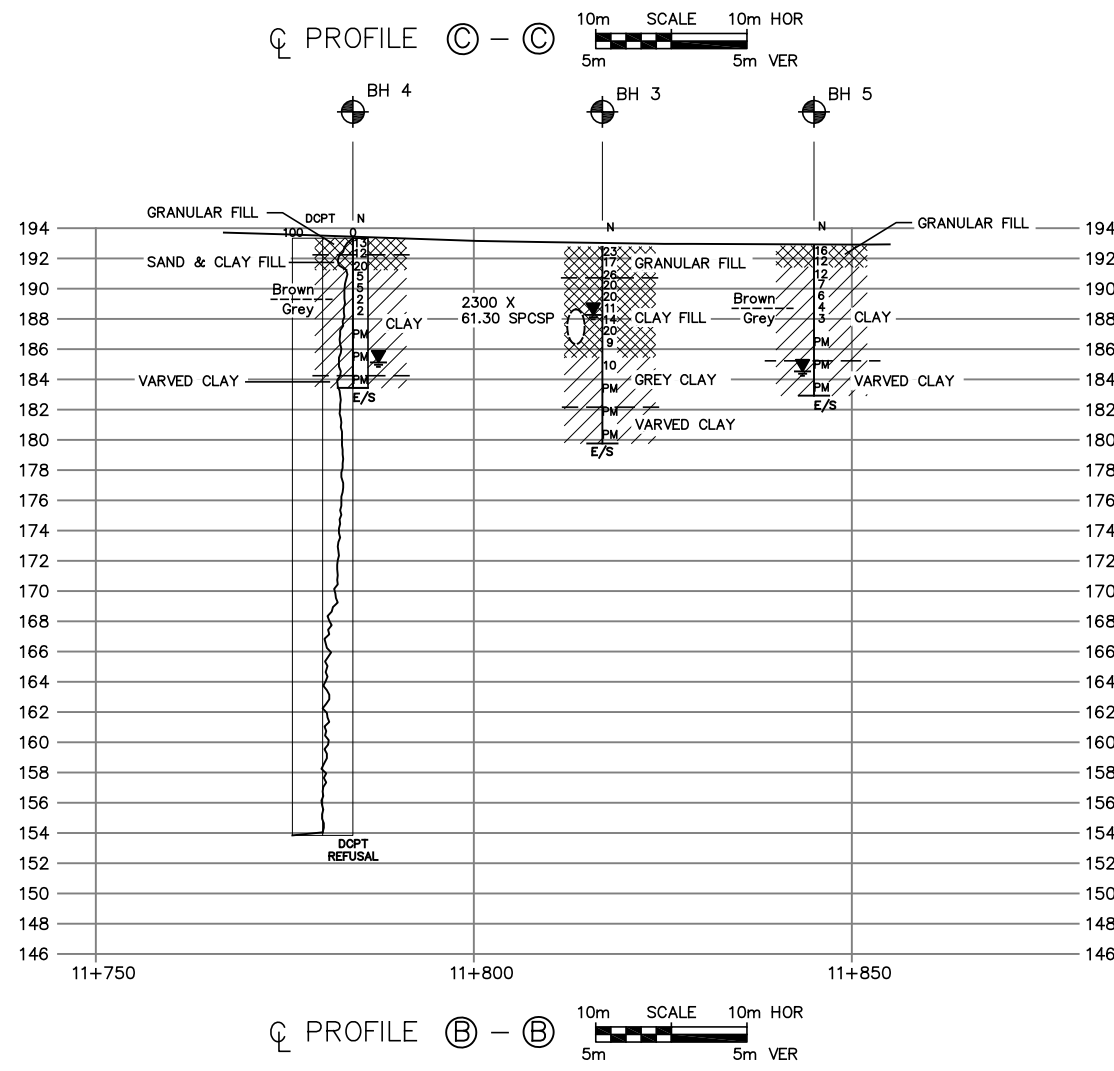
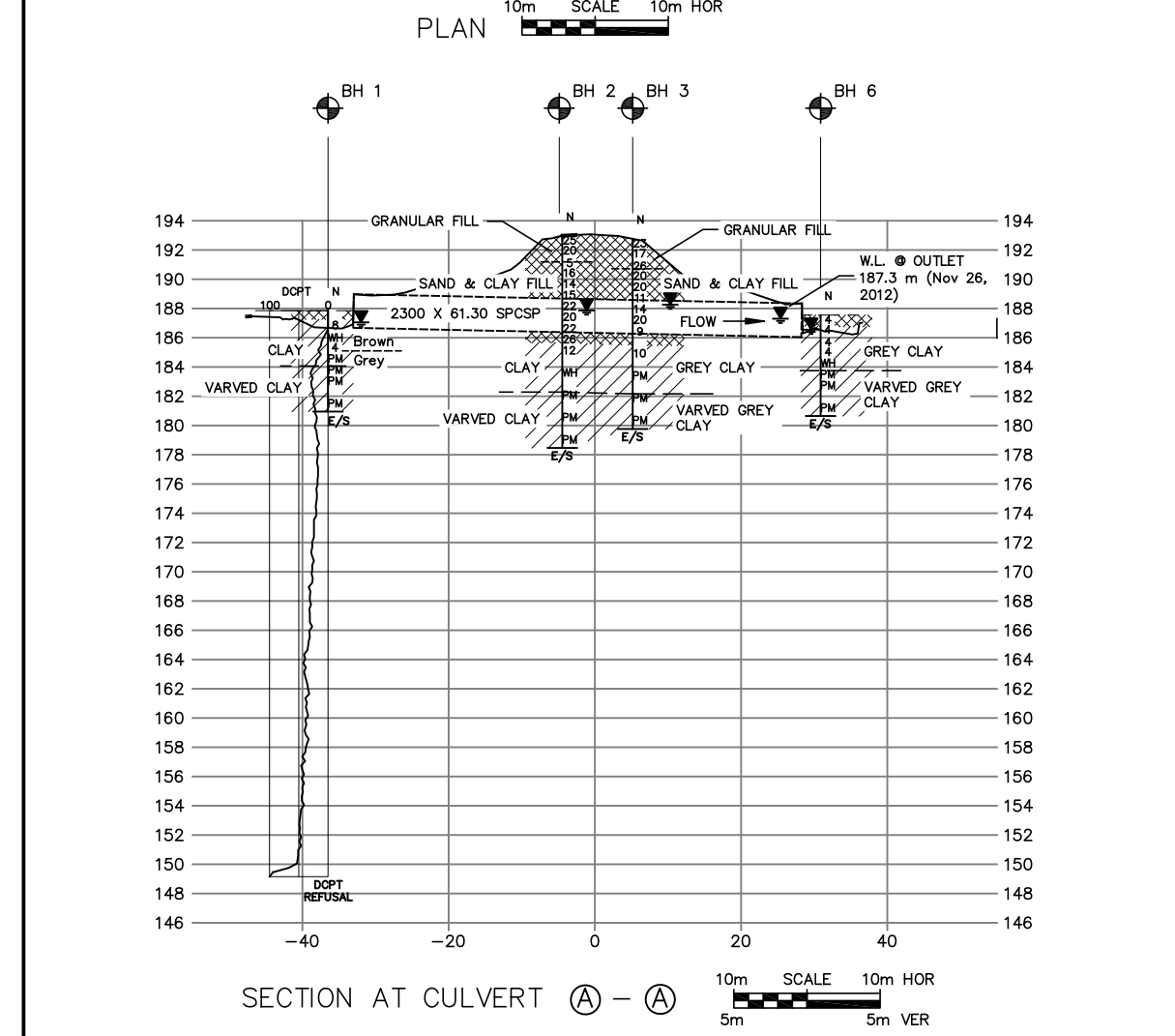
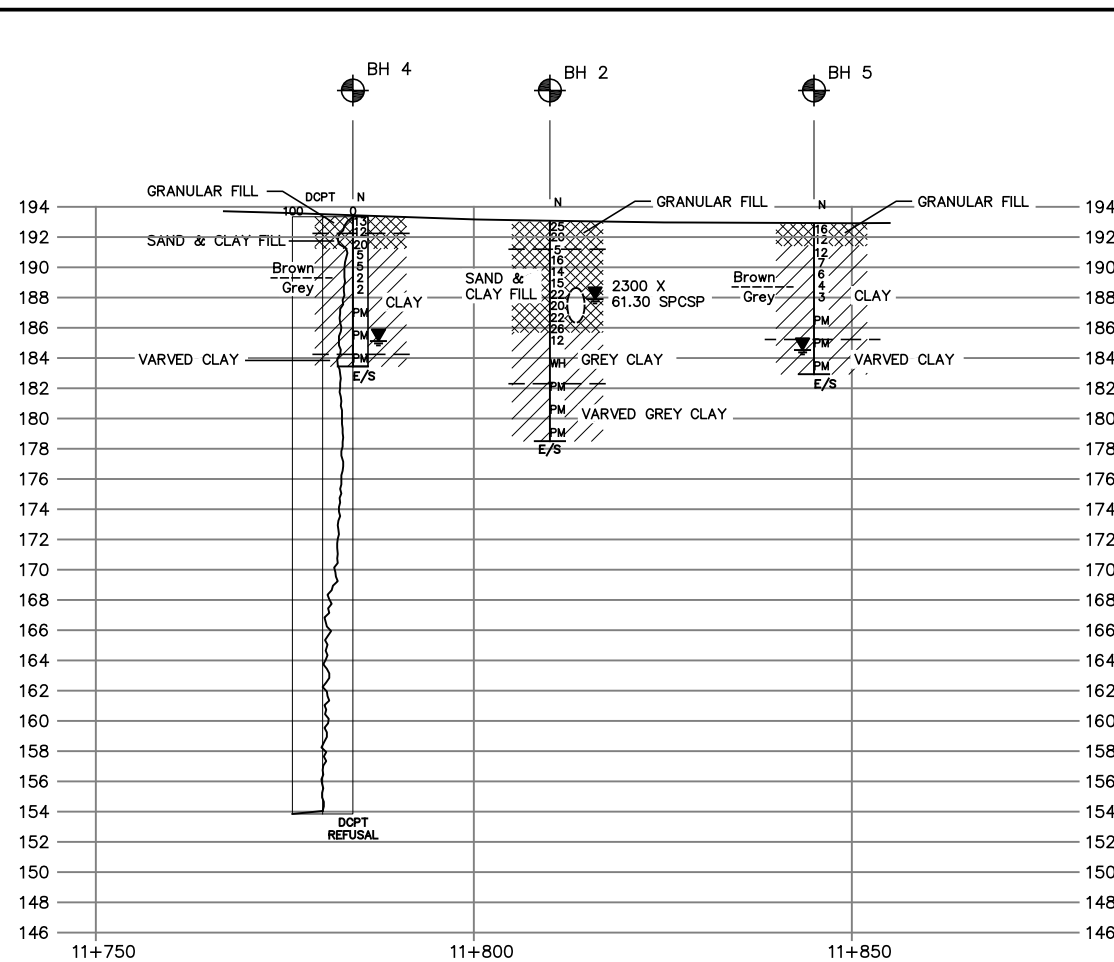
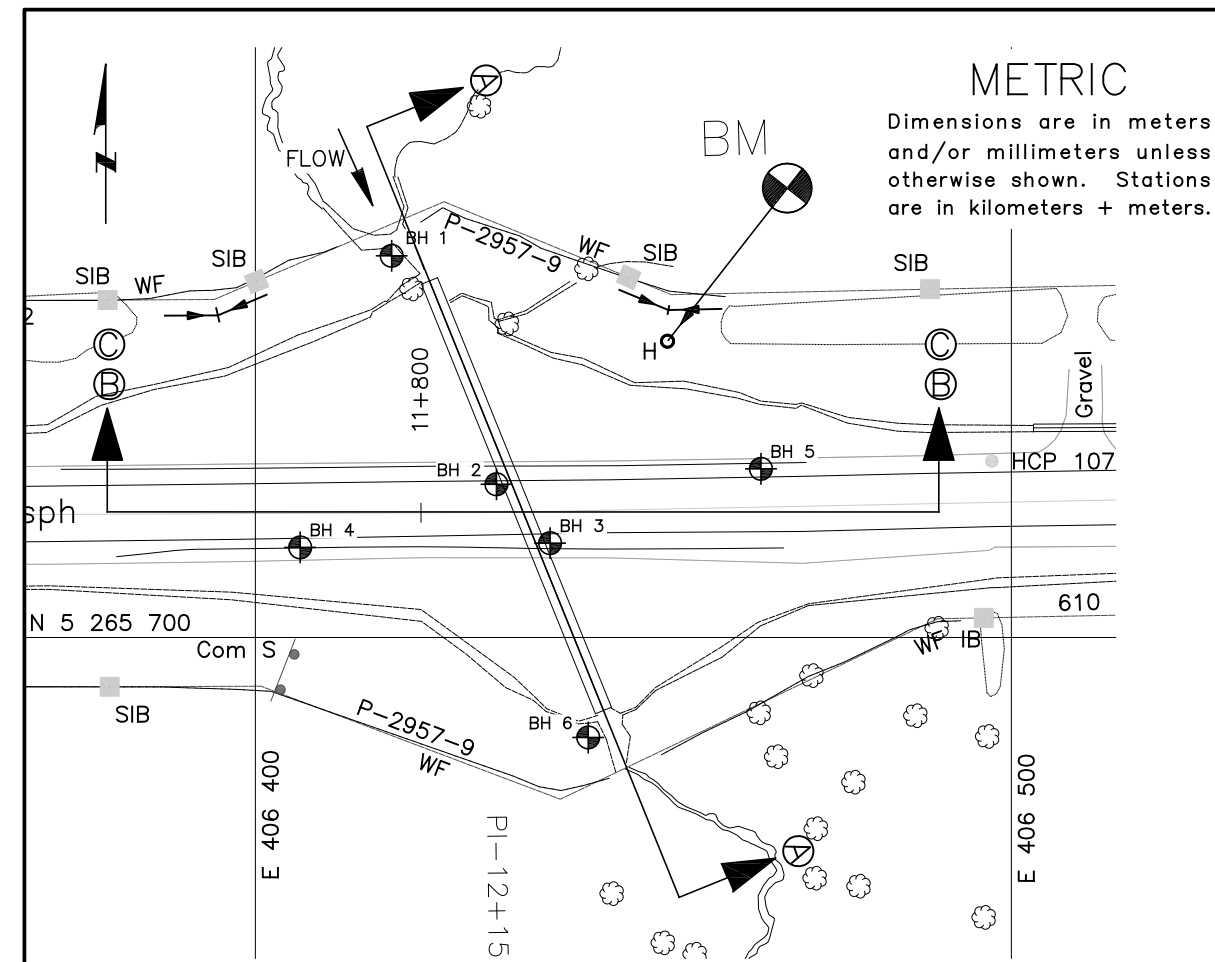
COMMENTS	+ 3, × 3 : Numbers on right refer to Sensitivity Numbers on left refer to values greater than 120 kPa ○ 3% STRAIN AT FAILURE	WATER LEVEL RECORDS		
		Date (dd/mm/yy)/Time	Water Depth (m)	Cave In (m)
The stratification lines represent approximate boundaries. The transition may be gradual.		1) 7/11/12 3:05:00 PM	5.2	▽ - 變
		2) 8/11/12 1:30:00 PM	1.8	▽ -
		3) 15/11/12 12:35:00 PM	1	▽ -

MEL-GEO 12028 - AREA A - BOREHOLE LOGS - FINAL.GPJ MEL-GEO.GDT 18/6/13

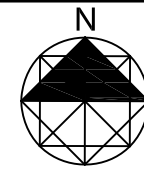


Appendix 3 Borehole Plan and Lab Data

Drawing No. 2: Borehole Location and Soil Strata
Figure Nos. L-1 to L-4: Grain Size Distribution Curves
Figure No. L-5: Atterberg Limits Sheet
Figure No. L-6: Shear Strength Chart
Figure Nos. L-7 and L-8: Consolidation Test Results
Figure No. L-9: Lab Test Summary Sheet



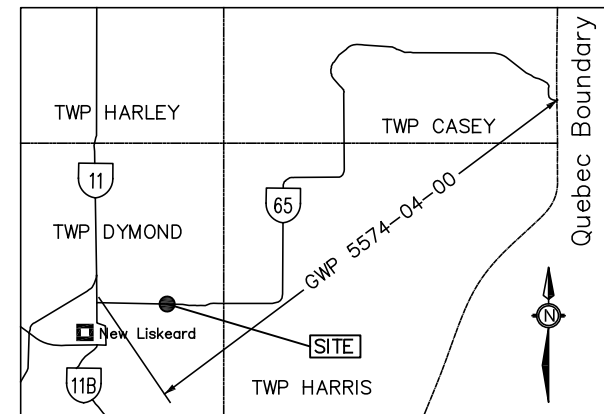
CONT No XXXX-XXXX
GWP No 5574-04-01



HWY NO. 65
Township of Dymond
Culvert at Station 11+814
BOREHOLE LOCATIONS & SOIL STRATA

Drawing
2

LVM | MERLEX



KEY PLAN — NOT TO SCALE
LEGEND

- Borehole
- Dynamic Cone Penetration Test (DCPT)
- Borehole and DCPT
- 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

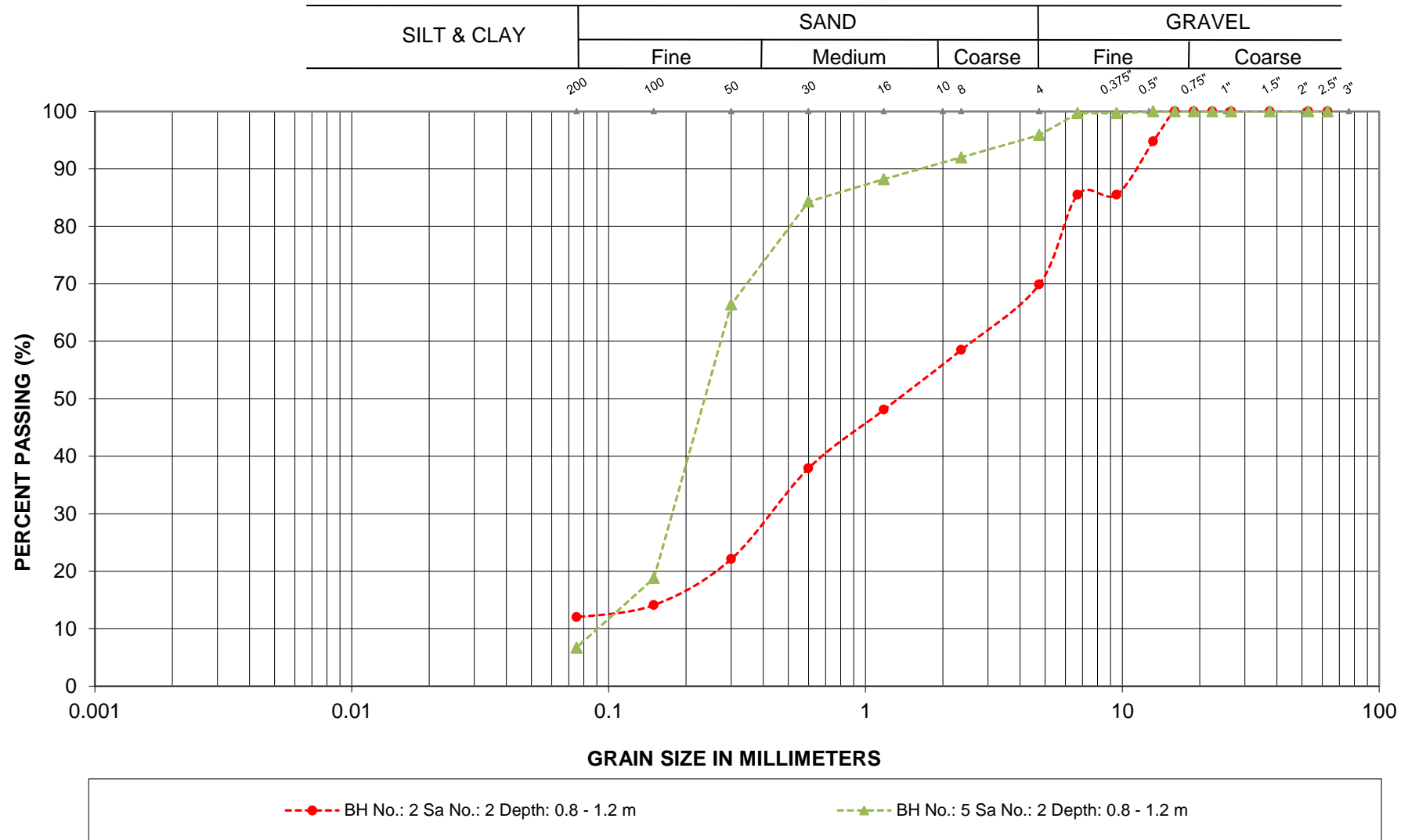
Borehole No.	Elev.	O/S	Co-ordinates	
			Northerly	Easterly
Borehole No. 1	187.9	34m Lt	5265751.3	406421.8
Borehole No. 2	193.0	3.7m Lt	5265720.3	406431.9
Borehole No. 3	192.8	4.2m Rt	5265712.5	406439.0
Borehole No. 4	193.3	4.5m Rt	5265711.9	406405.9
Borehole No. 5	192.8	5.0m Lt	5265722.3	406466.9
Borehole No. 6	187.6	30m Rt	5265686.8	406444.0

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	
	Jan 2013	RG	DRAFT	
	Apr 2013	RG	FINAL	
HWY No. 65 – Dymond Twp – Cvt at Sta. 11+814 LVM REF 12028–F1				
SUBM'D			GEOCRES 31M–104	SITE
DRAWN RG		CHK MAM	DATE December 2013	DWG 2

GRAIN SIZE ANALYSIS



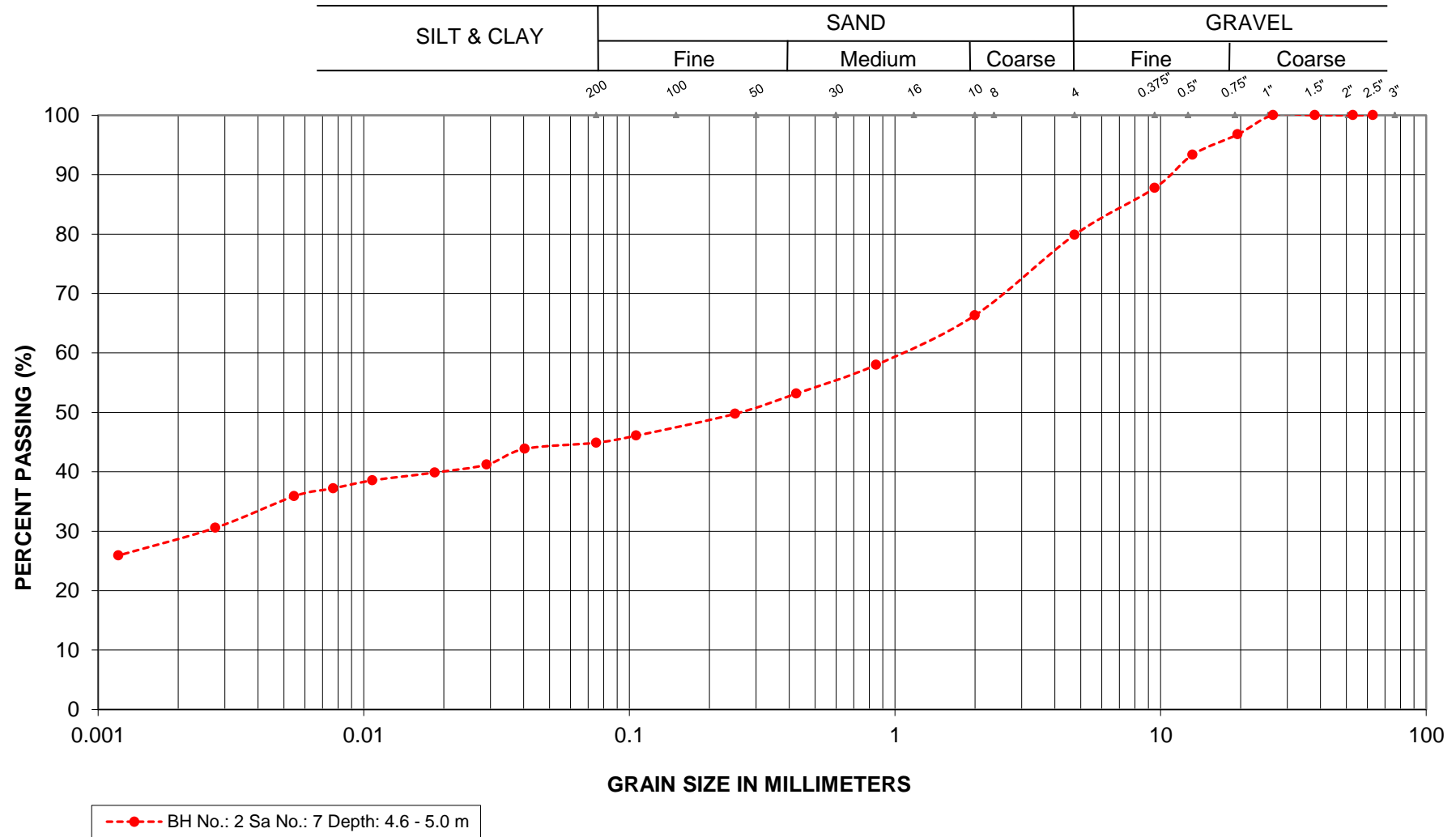
G.W.P.: 5574-04-00
LOCATION: Hwy 65

GRANULAR FILL

LVM | MERLEX

FIGURE L-1

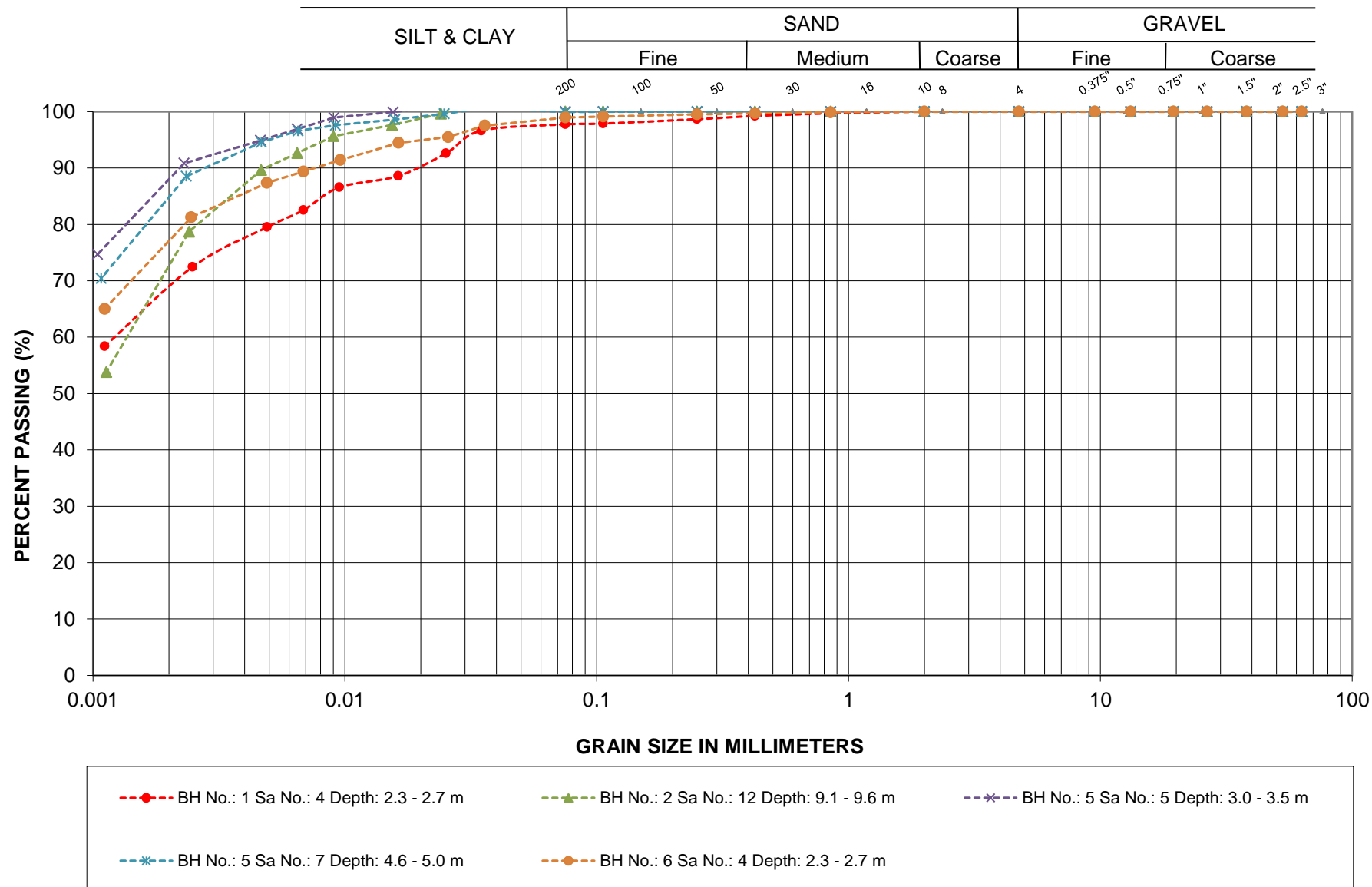
GRAIN SIZE ANALYSIS



G.W.P.: 5574-04-00
LOCATION: Hwy 65

SAND AND CLAY FILL

GRAIN SIZE ANALYSIS



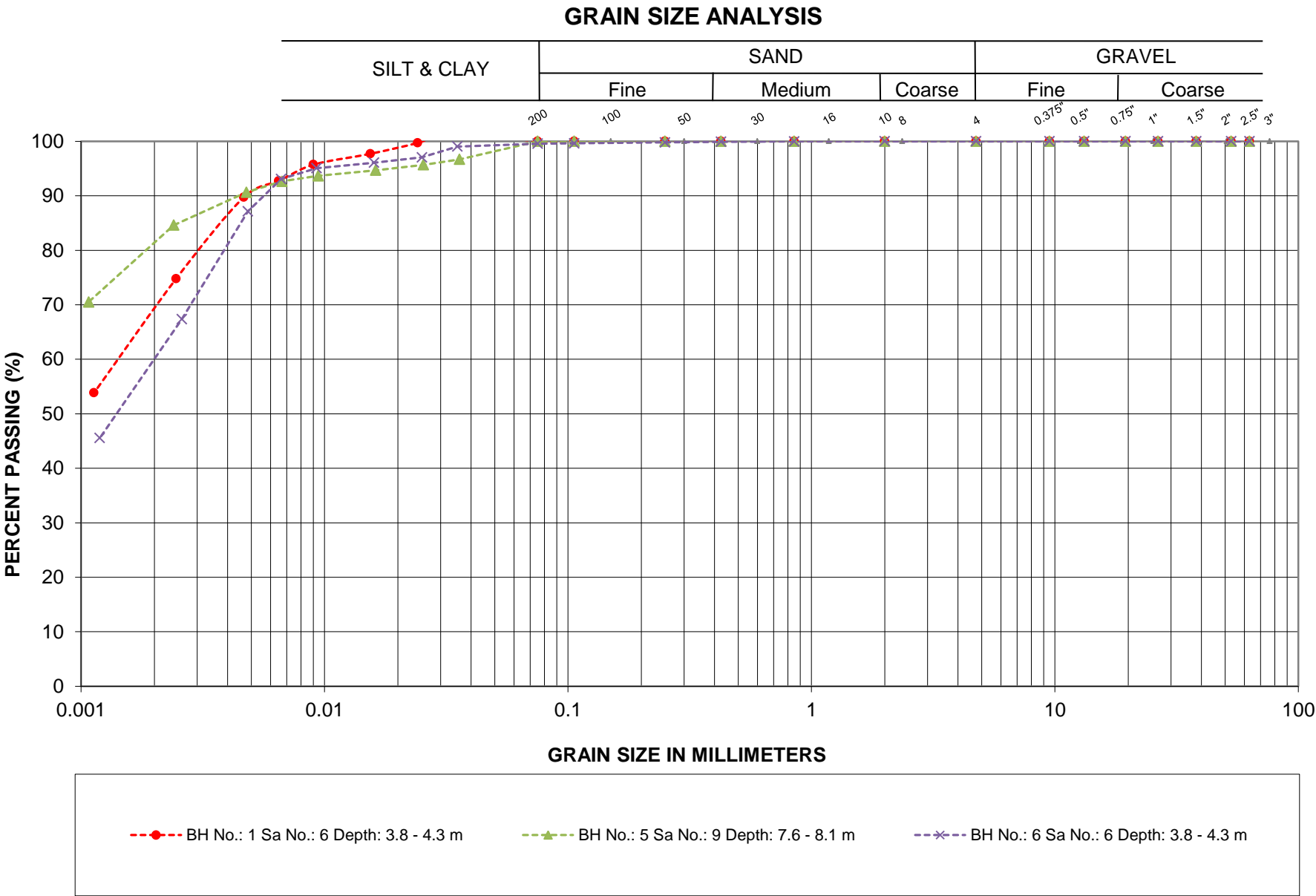
G.W.P.: 5574-04-00

LOCATION: Hwy 65

CLAY

LVM | MERLEX

FIGURE L-3



G.W.P.: 5574-04-00
LOCATION: Hwy 65

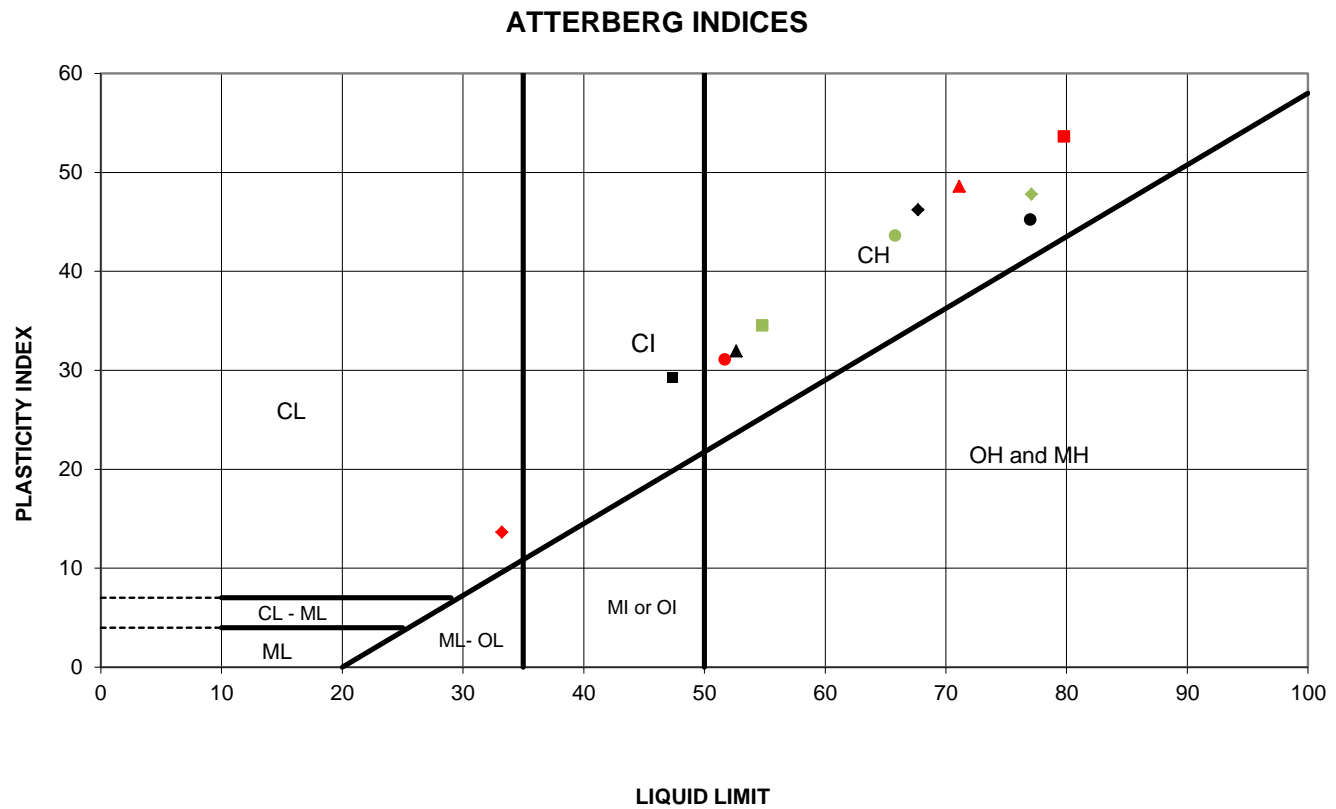
VARVED CLAY

LVM | MERLEX

FIGURE L-4

ATTERBERG LIMITS TEST RESULTS

FIGURE L-5



SYMBOL	BH	Sa. No.	Depth(m)	Elev.(m)	Liquid Limit	Plastic Limit	Plasticity Index	NMC %
●	1	4	2.3	185.6	77.0	31.8	45.2	43.2
◆	1	6	3.8	184.1	67.7	21.5	46.2	62.4
■	2	7	4.6	188.4	47.4	18.1	29.3	16.7
▲	2	12	9.1	183.9	52.6	20.7	31.9	50.5
●	2	14a	12.2	180.8	51.7	20.6	31.1	61.7
◆	2	14b	12.2	180.8	33.2	19.6	13.6	36.4
■	5	5	3.0	189.8	79.8	26.2	53.6	35.0
▲	5	7	4.6	188.2	71.1	22.5	48.6	44.9
●	5	9	7.6	185.2	65.8	22.2	43.6	52.3
◆	6	4	2.3	185.3	77.1	29.3	47.8	45.6
■	6	6	3.8	183.8	54.8	20.3	34.5	51.5

Date: Jun-13

Project: Hwy 65

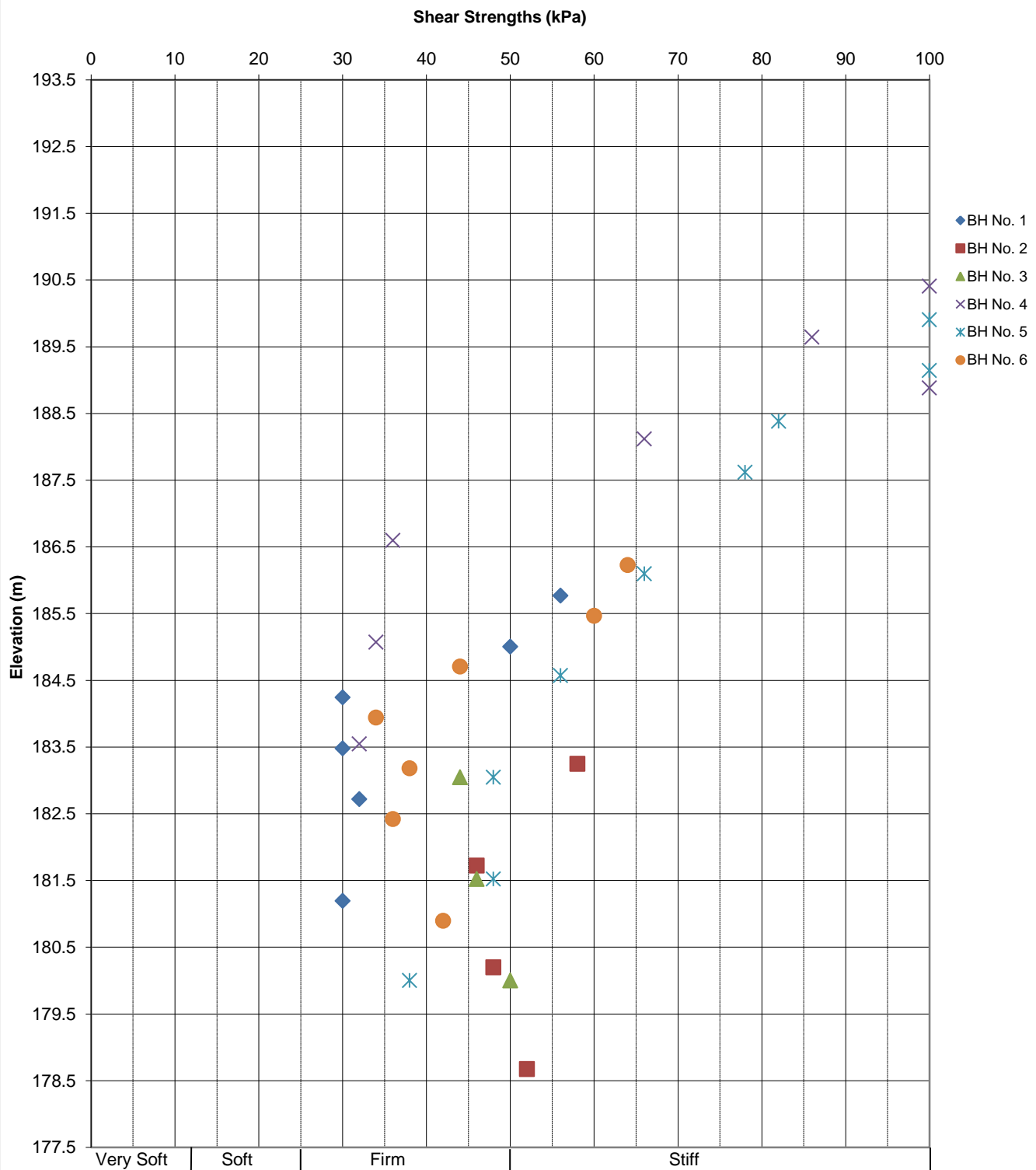
G.W.P: 5574-04-00

Prep'd: AT

Chkd: MAM

Ref. No.: 12/03/12028-F1

In-Situ Shear Strengths vs. Depth



CONSOLIDATION TEST SUMMARY**FIGURE L-7a****SAMPLE IDENTIFICATION**

Project Number	12-1183-0124	Sample Number	7
Borehole Number	1	Sample Depth, m	4.6

TEST CONDITIONS

Test Type	Standard	Load Duration, hr	24
Oedometer Number	8		
Date Started	12/21/2012		
Date Completed	01/04/2013		

SAMPLE DIMENSIONS AND PROPERTIES - INITIAL

Sample Height, cm	1.90	Unit Weight, kN/m ³	16.38
Sample Diameter, cm	6.35	Dry Unit Weight, kN/m ³	10.37
Area, cm ²	31.64	Specific Gravity, measured	2.73
Volume, cm ³	60.11	Solids Height, cm	0.736
Water Content, %	57.92	Volume of Solids, cm ³	23.29
Wet Mass, g	100.42	Volume of Voids, cm ³	36.82
Dry Mass, g	63.59	Degree of Saturation, %	100.0

TEST COMPUTATIONS

Stress kPa	Corr. Height cm	Void Ratio	Average Height cm	t ₉₀ sec	cv. cm ² /s	mv m ² /kN	k cm/s
0.00	1.900	1.581	1.900				
6.45	1.882	1.557	1.891	772	9.82E-04	1.44E-03	1.38E-07
11.08	1.871	1.542	1.877	1848	4.04E-04	1.26E-03	5.00E-08
21.12	1.854	1.518	1.863	1127	6.53E-04	9.07E-04	5.80E-08
40.53	1.826	1.480	1.840	2089	3.44E-04	7.59E-04	2.56E-08
79.19	1.766	1.398	1.796	1042	6.56E-04	8.24E-04	5.30E-08
156.51	1.661	1.256	1.713	1215	5.12E-04	7.12E-04	3.57E-08
311.32	1.531	1.080	1.596	1162	4.65E-04	4.42E-04	2.01E-08
620.59	1.428	0.940	1.480	694	6.69E-04	1.75E-04	1.15E-08
1239.14	1.342	0.823	1.385	470	8.65E-04	7.35E-05	6.23E-09
2478.39	1.265	0.719	1.304	227	1.59E-03	3.25E-05	5.05E-09
1239.14	1.272	0.727	1.269				
311.32	1.298	0.763	1.285				
79.19	1.328	0.804	1.313				
21.12	1.369	0.860	1.349				
6.45	1.398	0.899	1.384				

Note:

k calculated using cv based on t₉₀ values.

Consolidation loading schedule assigned by the client.

Specimen taken 18cm from the top of the tube.

SAMPLE DIMENSIONS AND PROPERTIES - FINAL

Sample Height, cm	1.40	Unit Weight, kN/m ³	18.87
Sample Diameter, cm	6.35	Dry Unit Weight, kN/m ³	14.10
Area, cm ²	31.64	Specific Gravity, measured	2.73
Volume, cm ³	44.23	Solids Height, cm	0.736
Water Content, %	33.87	Volume of Solids, cm ³	23.29
Wet Mass, g	85.13	Volume of Voids, cm ³	20.94
Dry Mass, g	63.59		

Prepared By: LFG

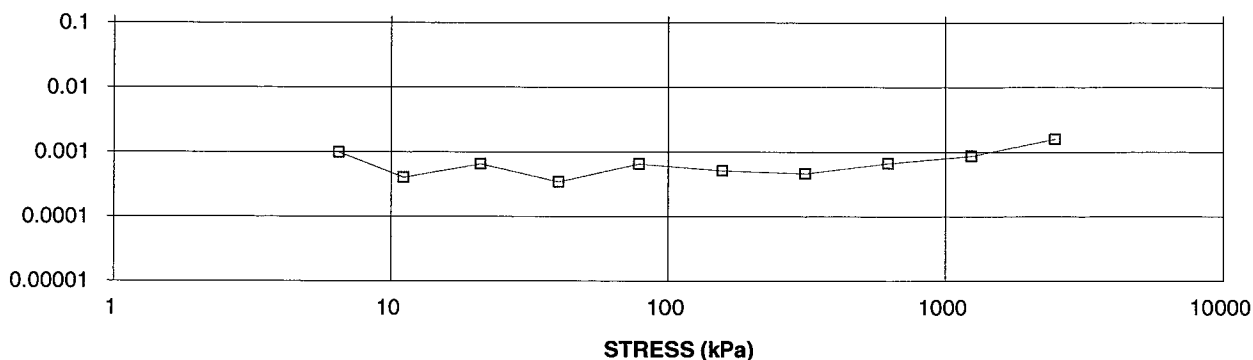
Golder AssociatesChecked By: 

CONSOLIDATION TEST SUMMARY

FIGURE L-7b

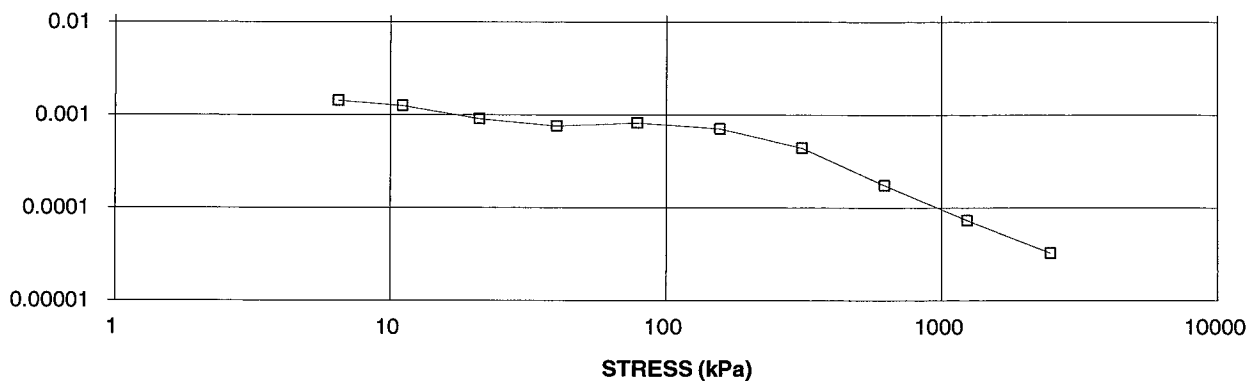
COEFFICIENT OF CONSOLIDATION,
cm²/s

CONSOLIDATION TEST
CV cm²/s VS STRESS (kPa)
BH 1 SA 7



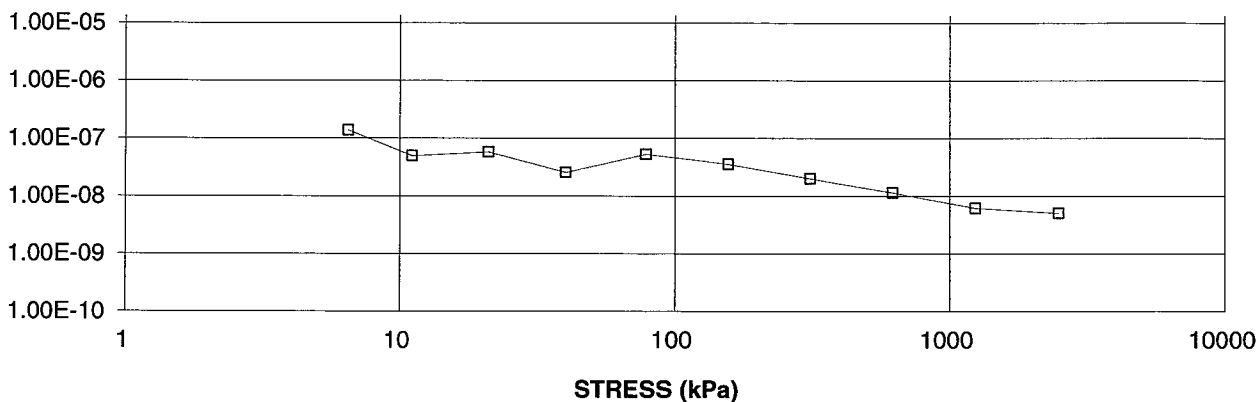
VOLUME COMPRESSIBILITY, m²/kN

CONSOLIDATION TEST
MV m²/kN vs STRESS (kPa)
BH 1 SA 7



HYDRAULIC CONDUCTIVITY,
cm/s

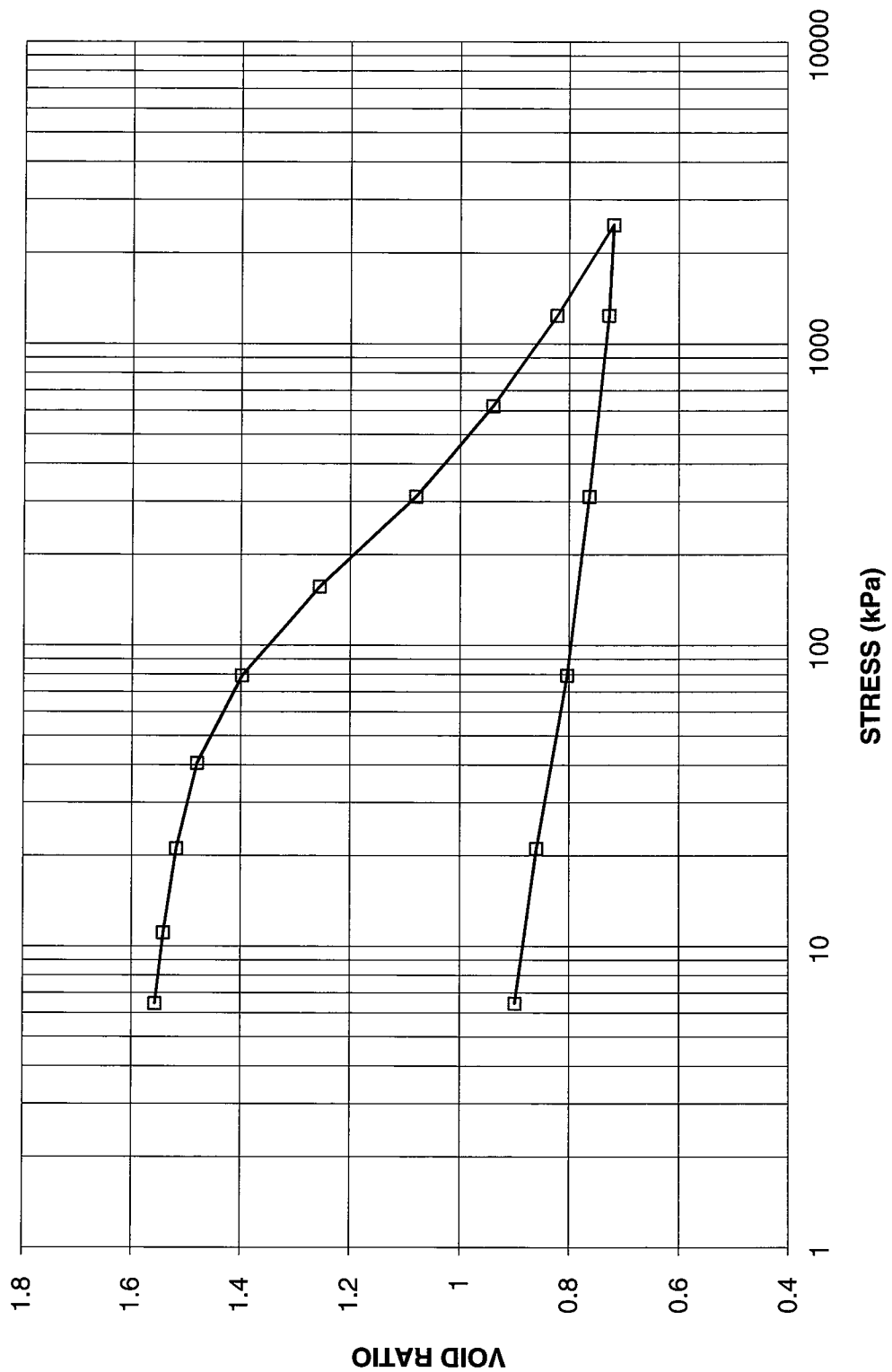
CONSOLIDATION TEST
HYDRAULIC CONDUCTIVITY vs STRESS
BH 1 SA 7



CONSOLIDATION TEST VOID RATIO VS LOG STRESS

FIGURE L-7c

CONSOLIDATION TEST
VOID RATIO vs STRESS
BH 1 SA 7



CONSOLIDATION TEST SUMMARY**FIGURE L-8a****SAMPLE IDENTIFICATION**

Project Number	12-1183-0124	Sample Number	11
Borehole Number	3	Sample Depth, m	10.7

TEST CONDITIONS

Test Type	Standard	Load Duration, hr	24
Oedometer Number	2		
Date Started	12/21/2012		
Date Completed	1/7/2013		

SAMPLE DIMENSIONS AND PROPERTIES - INITIAL

Sample Height, cm	2.53	Unit Weight, kN/m ³	16.29
Sample Diameter, cm	6.35	Dry Unit Weight, kN/m ³	10.23
Area, cm ²	31.71	Specific Gravity, measured	2.70
Volume, cm ³	80.26	Solids Height, cm	0.978
Water Content, %	59.26	Volume of Solids, cm ³	31.00
Wet Mass, g	133.32	Volume of Voids, cm ³	49.25
Dry Mass, g	83.71	Degree of Saturation, %	100.7

TEST COMPUTATIONS

Stress kPa	Corr. Height cm	Void Ratio	Average Height cm	t ₉₀ sec	cv. cm ² /s	mv m ² /kN	k cm/s
0.00	2.531	1.589	2.531				
5.87	2.524	1.582	2.528	746	1.82E-03	4.64E-04	8.26E-08
10.70	2.504	1.561	2.514	2233	6.00E-04	1.61E-03	9.48E-08
20.44	2.481	1.537	2.493	1815	7.26E-04	9.49E-04	6.75E-08
39.81	2.435	1.490	2.458	2160	5.93E-04	9.40E-04	5.46E-08
78.47	2.369	1.422	2.402	1852	6.60E-04	6.78E-04	4.38E-08
155.57	2.254	1.305	2.311	2693	4.20E-04	5.89E-04	2.43E-08
310.02	2.066	1.113	2.160	3985	2.48E-04	4.80E-04	1.17E-08
618.65	1.909	0.953	1.988	2018	4.15E-04	2.01E-04	8.16E-09
1236.18	1.781	0.821	1.845	1185	6.09E-04	8.22E-05	4.90E-09
2472.19	1.670	0.708	1.725	1567	4.03E-04	3.55E-05	1.40E-09
1236.18	1.677	0.715	1.673				
310.02	1.727	0.766	1.702				
78.47	1.782	0.822	1.754				
20.44	1.833	0.875	1.807				
5.87	1.877	0.920	1.855				

Note:

k calculated using cv based on t₉₀ values.

Consolidation loading schedule assigned by the client.

Specimen taken 10cm from the bottom of the tube.

SAMPLE DIMENSIONS AND PROPERTIES - FINAL

Sample Height, cm	1.88	Unit Weight, kN/m ³	18.74
Sample Diameter, cm	6.35	Dry Unit Weight, kN/m ³	13.79
Area, cm ²	31.71	Specific Gravity, measured	2.70
Volume, cm ³	59.52	Solids Height, cm	0.978
Water Content, %	35.89	Volume of Solids, cm ³	31.00
Wet Mass, g	113.75	Volume of Voids, cm ³	28.52
Dry Mass, g	83.71		

Prepared By: LFG

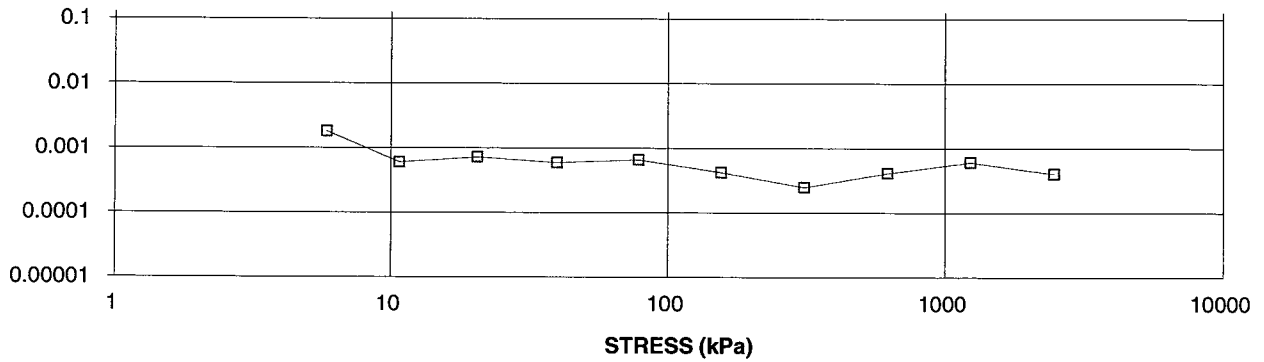
Golder AssociatesChecked By: 

CONSOLIDATION TEST SUMMARY

FIGURE L-8b

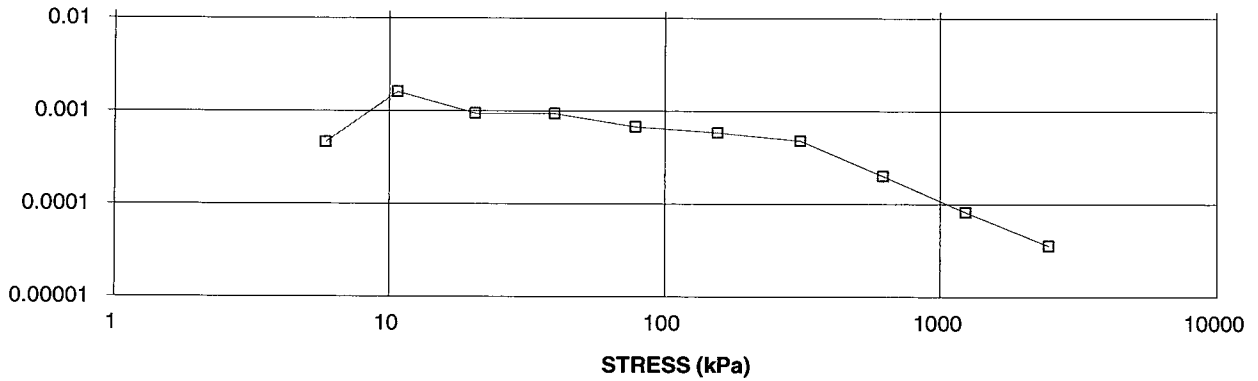
COEFFICIENT OF CONSOLIDATION,
cm²/s

CONSOLIDATION TEST
CV cm²/s VS STRESS (kPa)
BH 3 SA 11



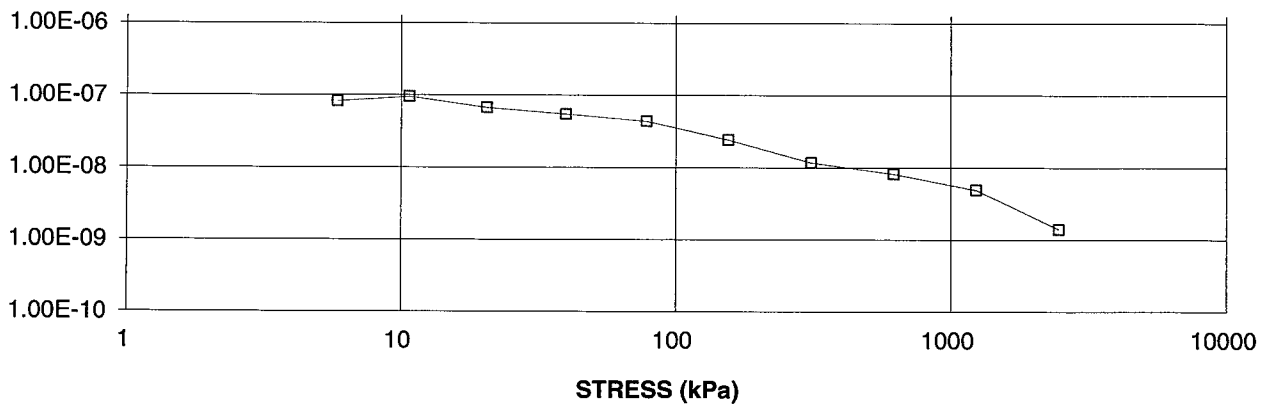
VOLUME COMPRESSIBILITY, m²/kN

CONSOLIDATION TEST
MV m²/kN vs STRESS (kPa)
BH 3 SA 11



HYDRAULIC CONDUCTIVITY,
cm/s

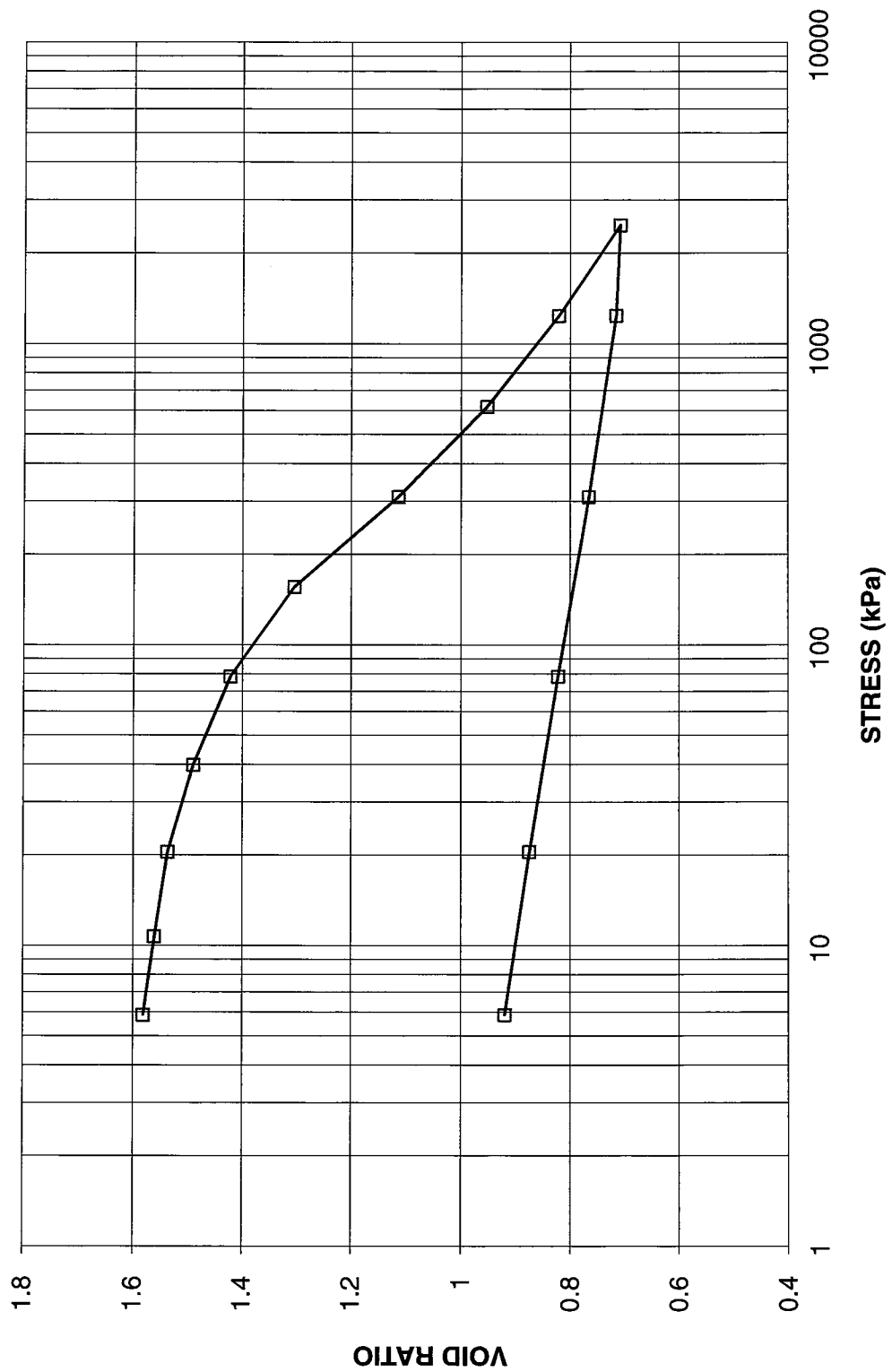
CONSOLIDATION TEST
HYDRAULIC CONDUCTIVITY vs STRESS
BH 3 SA 11



**CONSOLIDATION TEST
VOID RATIO VS LOG STRESS**

FIGURE L-8c

**CONSOLIDATION TEST
VOID RATIO vs STRESS
BH 3 SA 11**



Project No. 12-1183-0124

Prepared By: LFG

Golder Associates

Checked By: *[Signature]*

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					37.4				N/A			
	2	0.8					36.1				8			
	3	1.5					55.7				WH			
	4	2.3	0	2	29	69	43.2	77.0	31.8	45.2	4	CH		
	5	3.1					62.5				PM			
	6	3.8	0	0	30	70	62.4	67.7	21.5	46.2	PM	CH		
	7	4.6					31.7				PM		16.4	Consolidation Test
	8	6.1					64.8				PM			
2	1	0.0					4.2				25			
	2	0.8	30	58	12		6.0				20			
	3	1.5					32.5				5			
	4	2.3					15.6				16			
	5	3.1					24.7				14			
	6	3.8					26.2				15			
	7	4.6	20	35	17	28	16.7	47.4	18.1	29.3	22	CI		
	8	5.3					25.5				20			
	9	6.1					20.1				22			
	10	6.9					27.3				26			
	11	7.6					49.6				12			
	12	9.1	0	0	26	74	50.5	52.6	20.7	31.9	WH	CH		
	13	10.7					38.4				PM			
	14a	12.2					61.7	51.7	20.6	31.1	PM	CH		
	14b	12.2					36.4	33.2	19.6	13.6	PM	CI		
	15	13.7					45.8				PM			
3	1	0.0					5.7				23			
	2	0.8					3.7				17			
	3	1.5					8.4				26			

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	4	2.3					7.9				20			
	5	3.1					26.6				20			
	6	3.8					29.6				11			
	7	4.6					30.3				14			
	8	5.3					17.9				20			
	9	6.1					19.6				9			
	10	7.6					37.1				10			
	11	9.1					62.1				PM		16.3	Consolidation test
	12	10.7					42.1				PM			
	13	12.2					46.1				PM			
4	1	0.0					5.7				13			
	2	0.8					12.6				12			
	3	1.5					10.1				20			
	4	2.3					40.4				5			
	5	3.1					45.8				5			
	6	3.8					48.6				2			
	7	4.6					51.4				2			
	8	6.1					56.3				PM			
	9	7.6					61.9				PM			
	10	9.1					65.2				PM			
5	1	0.0					5.9				16			
	2	0.8	4	89		7	14.0				12			
	3	1.5					25.2				12			
	4	2.3					33.9				7			
	5	3.1	0	0	12	88	35.0	79.8	26.2	53.6	6	CH		
	6	3.8					45.1				4			
	7	4.6	0	0	15	85	44.9	71.1	22.5	48.6	3	CH		

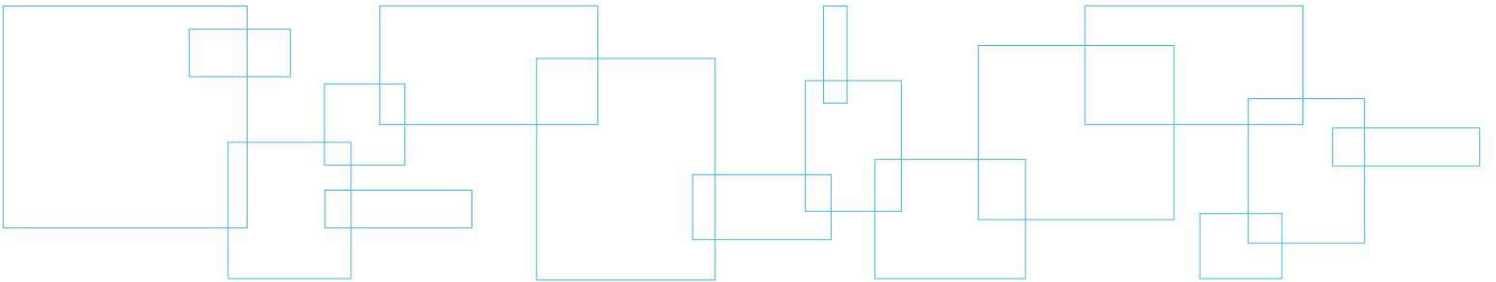
Laboratory Tests - Summary Sheet

[illegible]

Appendix 4 Photo Essay

Enclosure No. 8:

Photo Essay



Culvert Inlet – Looking North

Photo: 1



Culvert Inlet – Looking East

Photo: 2



Project: Hwy 65 – Station 11+814, Twp of Dymond

Photos Provided By: LVM

Date: November 2012

Culvert Outlet – Looking North

Photo: 3



View Through Culvert – Looking South

Photo: 4



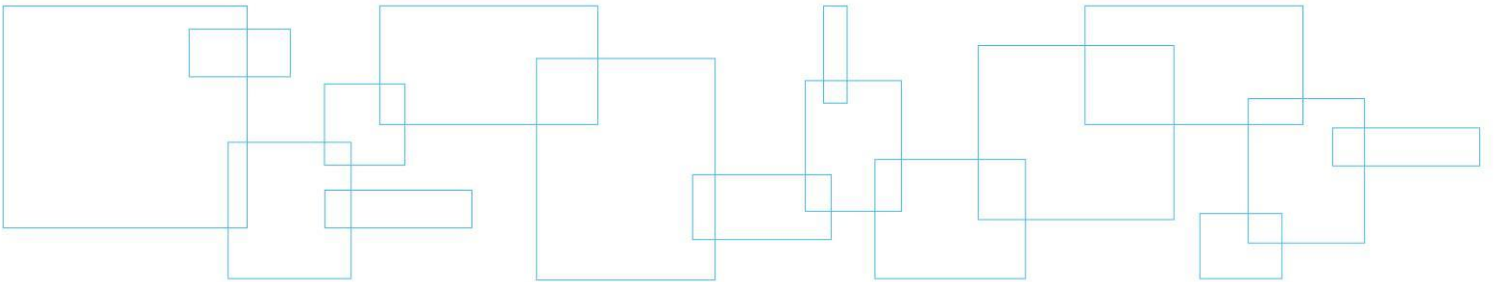
Project: Hwy 65 – Station 11+814, Twp of Dymond

Photos Provided By: LVM

Date: November 2012

Appendix 5 Historical Data

1961 Borehole Logs and Site Plan



DEPARTMENT OF HIGHWAYS - ONTARIO

MATERIALS AND RESEARCH SECTION

W.P. 26-61

BORE HOLE NO. 1

JOB 61-F-56

STATION 59+31 (12' R.L.)

DATUM 625.6'

COMPILED BY B.K.

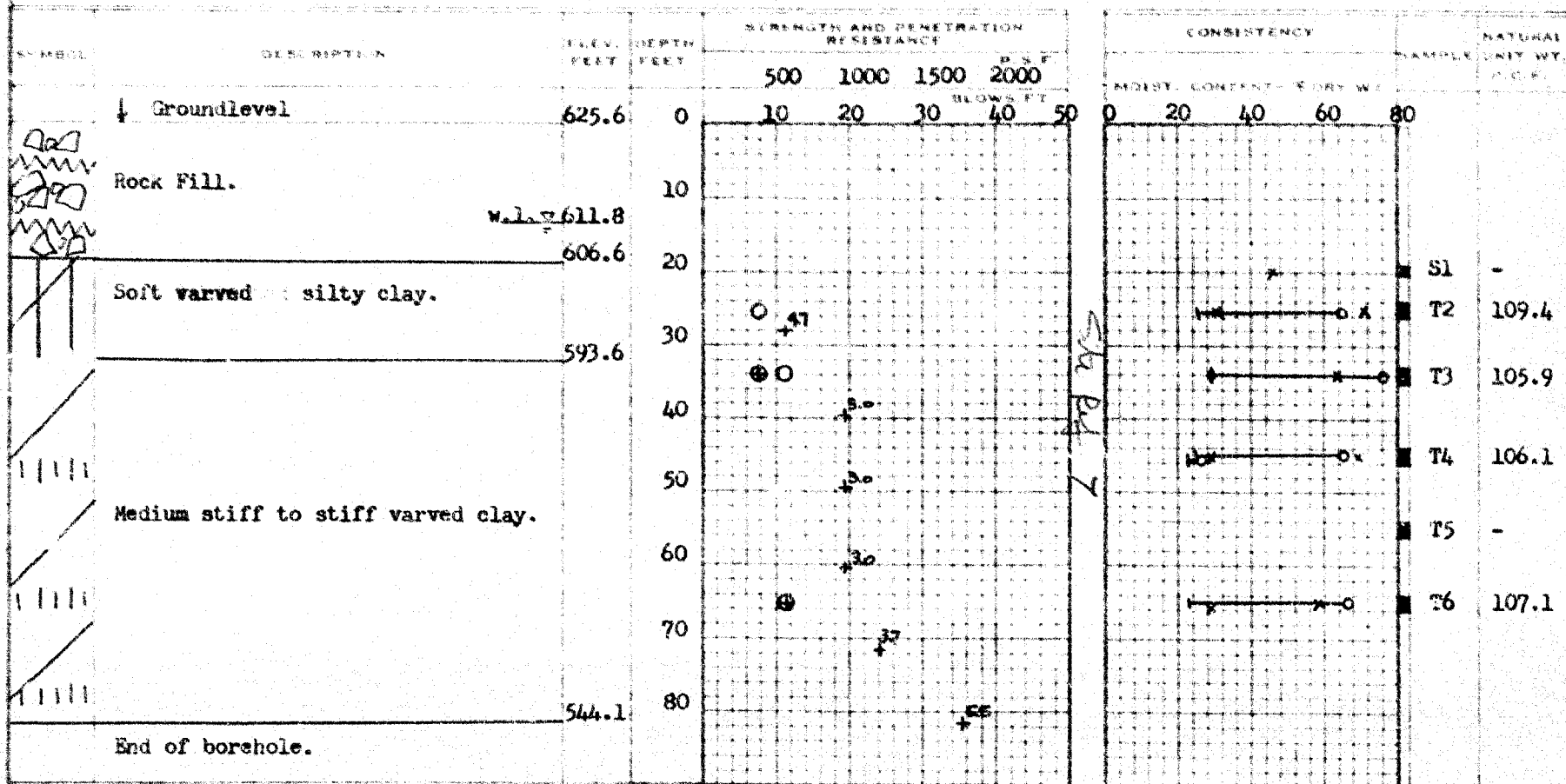
BORING DATE June 16/61

CHECKED BY R.W.K.

2" DIA. SPLIT TUBE
 2" SHELLEY TUBE
 2" SPLIT TUBE
 2" DIA. CONE
 2" SHELLEY
 CASING

LEGEND

Lab vane - - - - -
 1/2 UNCONFINED COMPRESSION (QU)
 VANE TEST (C) AND SENSITIVITY (S)
 NATURAL MOISTURE AND
 LIQUIDITY INDEX
 LIQUID LIMIT
 PLASTIC LIMIT



DEPARTMENT OF HIGHWAYS - ONTARIO

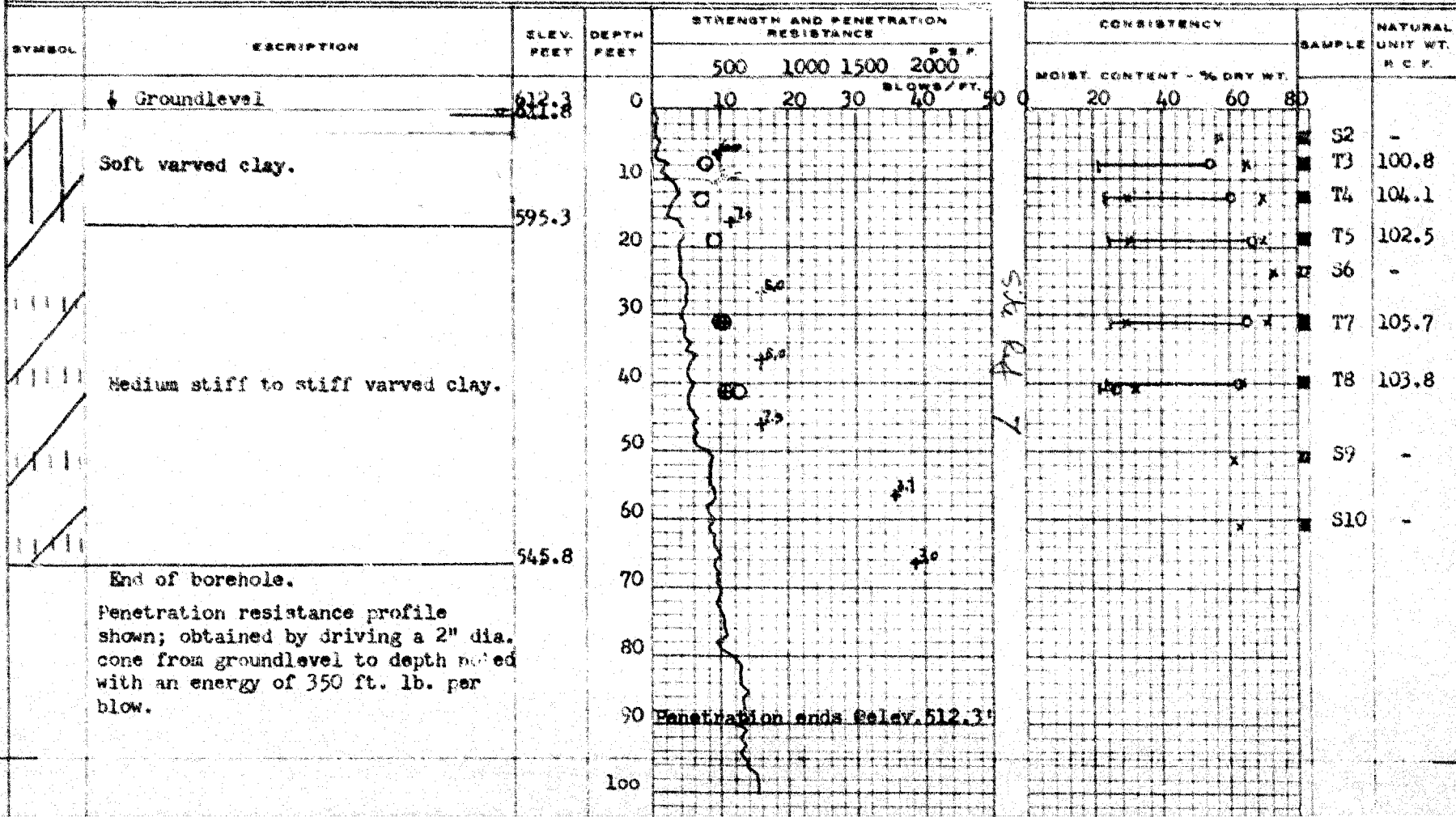
MATERIALS AND RESEARCH SECTION

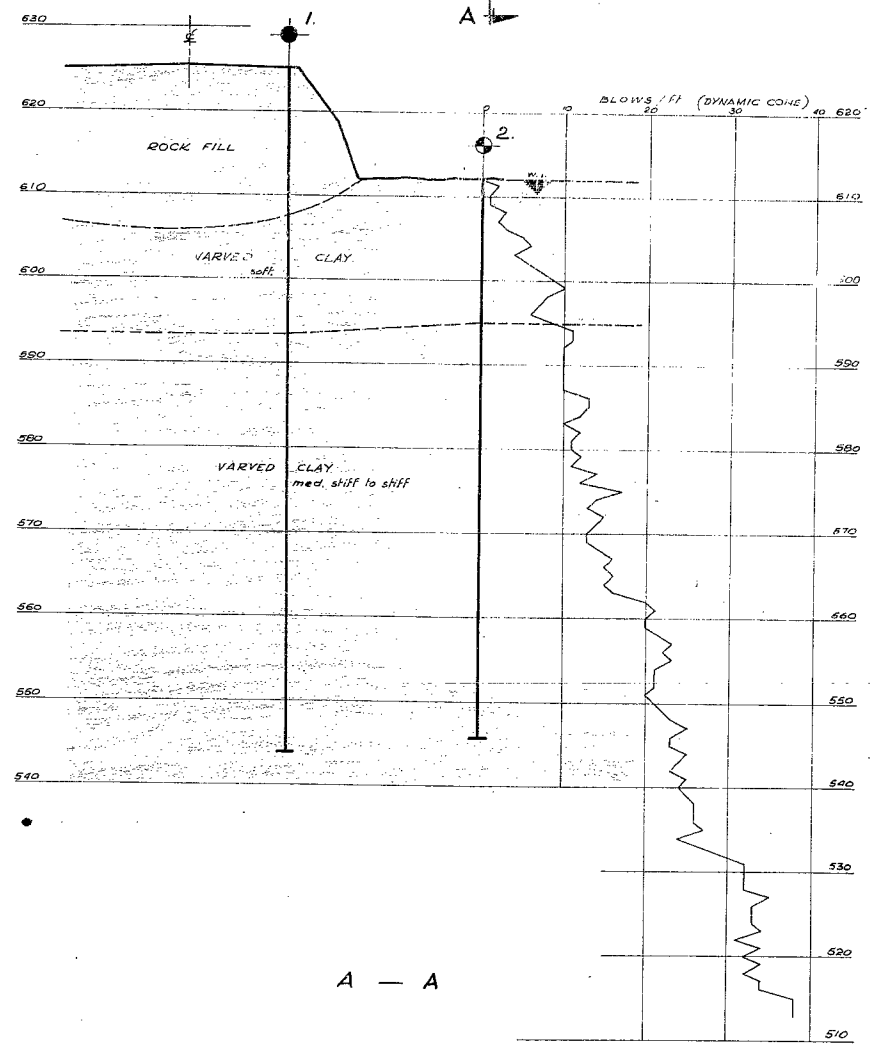
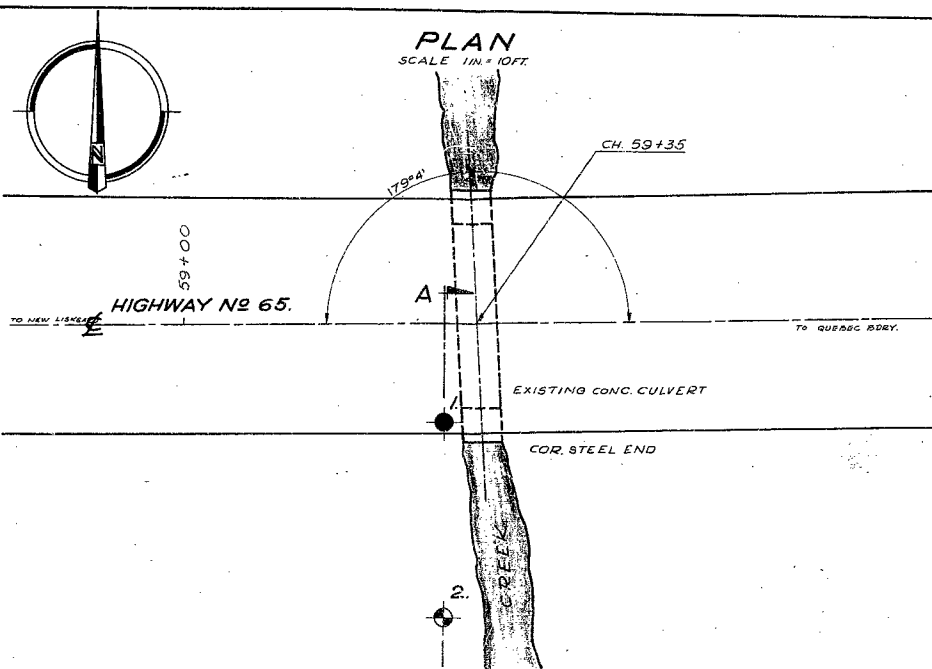
W.P. 24-61 BORE HOLE NO. 2
 JOB 61-F-56 STATION 59+31 (35' Rt.)
 DATUM 612.3' COMPILED BY B.K.
 BORING DATE JUNE 17/61 CHECKED BY W.W.K.

2" DIA SPLIT TUBE
 2" SHELBY TUBE
 2" SPLIT TUBE
 2" DIA CONE
 2" SHELBY
 CASING

LEGEND

Lab Vane
 1/2 UNCONFINED COMPRESSION (Qu)
 VANE TEST (C) AND SENSITIVITY (S)
 NATURAL MOISTURE AND
 LIQUIDITY INDEX
 LIQUID LIMIT
 PLASTIC LIMIT





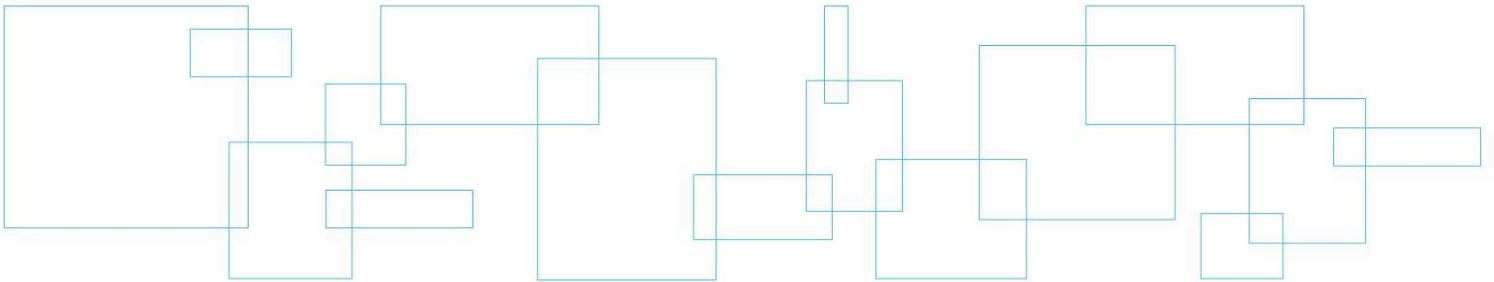
LEGEND			
	BORE HOLE		
	BORE AND PENETRATION HOLE		
HOLE	ELEVATION	STATION	OFFSET
1	625.6	59+31	12' RT.
2	612.3	59+31	35' RT.

5 59601400
5264000
17
31M 12 IE

DEPARTMENT OF HIGHWAYS - ONTARIO MATERIALS & RESEARCH SECTION			
FILL STABILITY AT HWY No 65			
CHAINAGE 59+31			
ORIGINATED <i>W. KULMATICAS</i>	DISTRICT NO. 14	DATE 31 AUG 1961	
DRAWN <i>T. K. K. 10/10/61</i>	W.P. NO. 26-61	JOB NO. 61-F-56	
CHECKED <i>W. K. 10/10/61</i>	SCALE 15 SHOWN	DRAWING NO.	
APPROVED <i>W. K. 10/10/61</i>		61-F-56 A	

Appendix 6 Reference Data

Enclosure No. 9 Geotechnical Borehole Data
Figure No. P-1 Clay Proctor Results
Non Standard Special Provision – Open Cut Excavations
Settlement Monitoring Guidelines



Highway 65
Cuvert Replacement
Station 11+814 – Twp. of Dymond
Geotechnical Borehole Logs

11+802 3.5 Rt C/L

0 - 80 Asph
80 - 230 Cr Gr
230 - 550 Cr Gr & RAP
550 - 2.6 F-Med Sa W Gr
 Occ Cob

11+806 3.5 Rt C/L

0 - 80 Asph
80 - 240 Cr Gr
240 - 600 Cr Gr & RAP
600 - 2.6 F-Med Sa W Gr Occ Cob

11+810 3.5 Rt C/L

0 - 90 Asph
90 - 260 Cr Gr
260 - 700 Cr Gr & RAP
700 - 2.6 F-Med Sa W Gr Occ Cob

11+814 3.5 Rt C/L

0 - 90 Asph
90 - 220 Cr Gr
220 - 500 Cr Gr & RAP
500 - 850 F-Med Sa W Gr
850 - 2.6 F-Med Sa Tr Gr
 Occ Cob

11+818 3.5 Rt C/L

0 - 90 Asph
90 - 250 Cr Gr
250 - 550 Cr Gr & RAP
550 - 900 F-Med Sa W Gr
900 - 2.6 F-Med Sa Tr Gr Occ Cob

11+822 3.5 Rt C/L

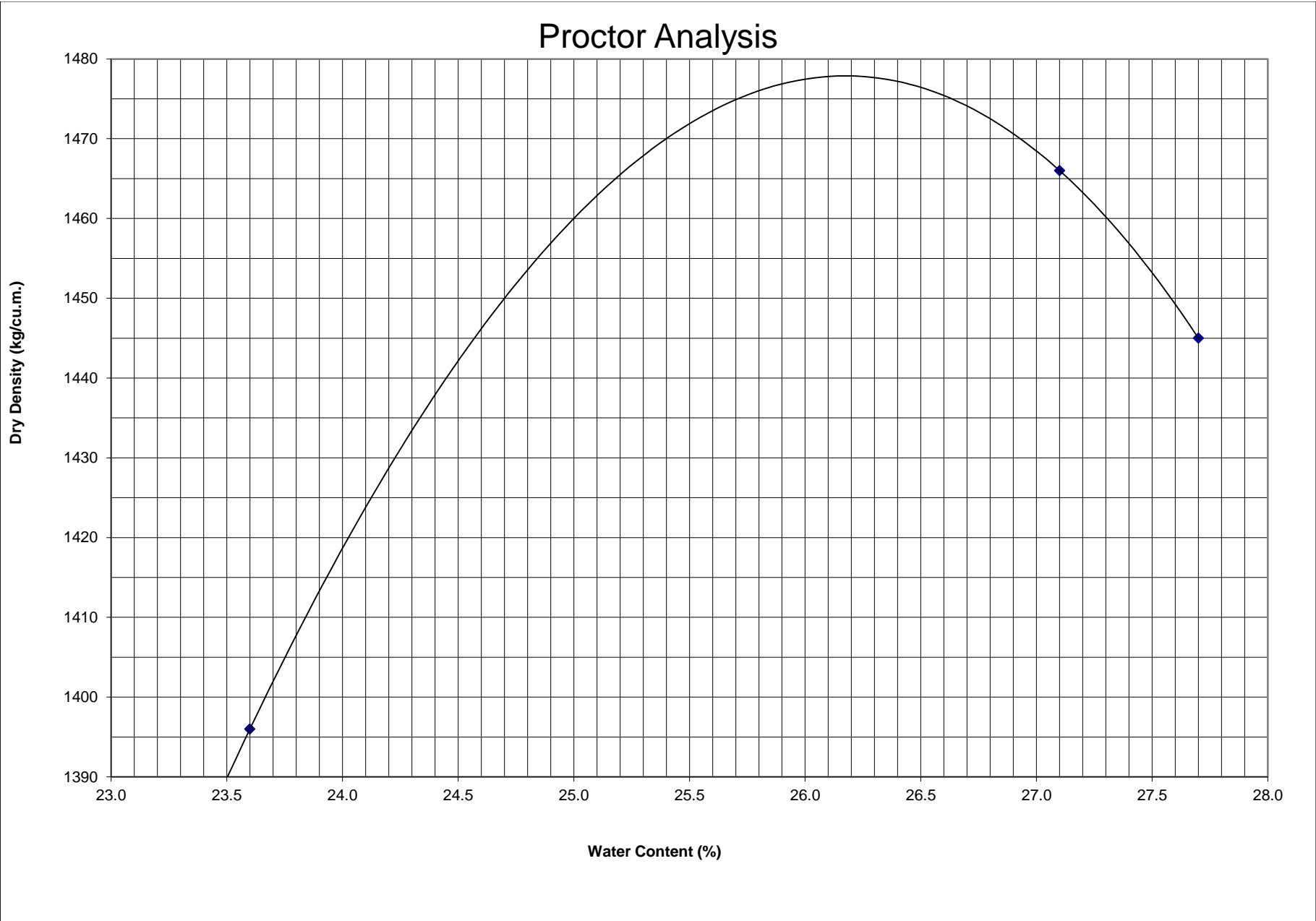
0 - 80 Asph
80 - 250 Cr Gr
250 - 500 Cr Gr & RAP
500 - 2.6 F-Med Sa W Gr Occ Cob

11+825 3.5 Rt C/L

0 - 80 Asph
80 - 250 Cr Gr
250 - 500 Cr Gr & RAP
500 - 2.6 F-Med Sa Tr Gr Occ Cob

11+836 2.2 Rt C/L

0 - 80 Asph
80 - 200 Cr Gr
200 - 450 Cr Gr & RAP
450 - 1.5 F-Med Sa Tr Gr



GWP: 5574-04-00
Location: Station 11+814, Twp of Dymond

CLAY PROCTOR

2700 and 2200 mm Pipe Culverts - Item No.

Special Provision

C/L Culvert Station 11+814

Temporary unsupported open cut excavations shall not have slopes steeper than 1.5H: 1V.

When the vertical height of the temporary unsupported open cut excavation is greater than 5 m and less than or equal to 7m, the pipe installation shall be backfilled, to the level of the pipe obvert, and brought up to the toe of the slope by shut down each day.

Temporary unsupported open cut excavations greater than 7 m in height shall not have slopes steeper than 2H:1V over the full slope height; Temporary unsupported open cut excavations greater than 7 m in height must be inspected by a geotechnical engineer on a daily basis; Prior to commencing an unsupported excavation greater than 7 m the geotechnical engineer must be given 24 hours' notice:

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.

C/L Culvert Station 12+548

Temporary unsupported open cut excavations shall not have slopes steeper than 1.5H: 1V.

When the vertical height of the temporary unsupported open cut excavation is greater than 6 m and less than or equal to 8 m, the pipe installation shall be backfilled, to the level of the pipe obvert, and brought up to the toe of the slope by shut down each day.

Temporary unsupported open cut excavations greater than 8 m in height shall not have slopes steeper than 2H:1V over the full slope height. Temporary unsupported open cut excavations greater than 8 m in height must be inspected by a geotechnical engineer on a daily basis. Prior to commencing an unsupported excavation greater than 8 m the geotechnical engineer must be given 24 hours' notice:

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.

APPENDIX: SETTLEMENT MONITORING GUIDELINES - TUNNELING

The purpose of settlement monitoring is to prevent damage to existing utilities and highway structures along the tunnel alignment. Ground settlement include settlement due to lost ground and dewatering/drainage.

Instrumentation Arrays

All measurement points shall be installed and surveyed before the start of excavation to establish benchmarks/baseline.

Surface Monitoring Points

Surface monitoring points will be installed to cover the whole length of the tunnel with in the right of way under the jurisdiction of MTO (Figure 1).

Surface monitoring points will be located at not greater than 5m intervals along the tunnel alignment. The surface monitoring will be identified using paint marks on the pavement. Surface monitoring points installed on the unpaved right of way shall be founded below frost penetration depths. The interval and/or marking of the points should be changed with MTO's approval where traffic disruptions might occur.

The final instrumentation plan should be finalised when Contractor's proposed construction method is available.

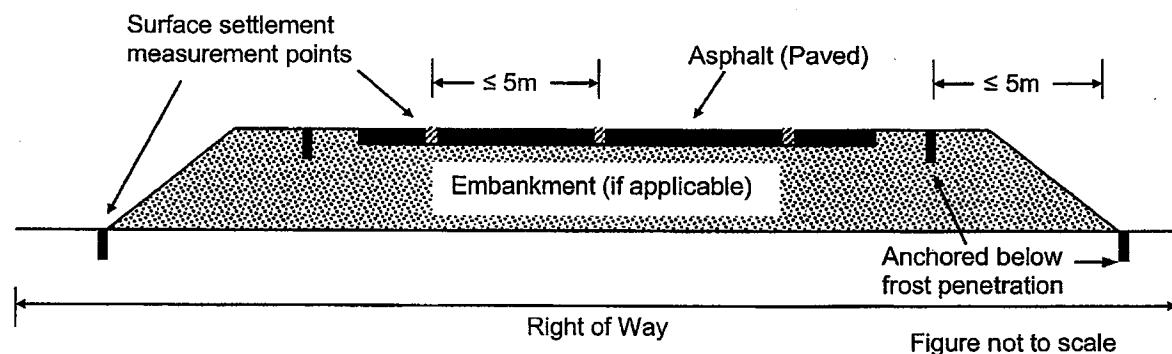


Figure 1: Typical configuration of surface settlement monitoring points along the tunnel alignment.

Condition Survey

A condition survey for the pavement will be carried out prior to commencement of construction and documented for the purpose of requirement of restoration. The condition survey shall document visible flaws such as cracks, distortions and deviations, heaves, and depressions. This surface survey will be completed during the installation of the monitors and again once the tunnel has been completed.

Reading Frequency

An average of at least two readings shall be taken to establish the initial conditions.

The reading and collection of data from the surface monitoring points shall be read and recorded by the Contractor during the construction period and after construction for period of at least 2 weeks provided that further settlement has stopped.

A minimum of three (3) sets of reading be taken daily, provided that movements are within anticipated limits. Otherwise, the frequencies should increase according to a pre-planned interval.

Monitoring of movements is required during work stoppages, such as during non-operation period (off-shifts) or weekends. A minimum of three (3) sets of readings should be taken daily.

Measurements of the monitoring points shall be reported promptly to MTO for review.

Data Collection and Data Transfer

A procedure is required to be established in consultation with MTO so that the monitoring data and the interpreted data will reach all parties as soon as necessary. The contract administrator/consultant and the Contractor should interpret monitoring data as needed for the purpose of on-going construction. The Foundation Engineer should be contacted for technical support to the prime Consultant in the interpretation of ground movements and review of the Contractor's response when Review and Alert Levels are reached.

Criteria for Assessment

The acceptable surface settlement (or heave) will be according to criteria as specified below.

Baseline Reading – A baseline reading of the instrumentation shall be taken prior to commencement of the work. An average of at least two initial readings shall be recorded as baseline reading.

Review Level – A maximum value of 10 mm relative to the baseline readings is suggested for this project. If this level is reached, the method, rate or sequence of construction, or ground stabilization measures should be reviewed or modified to mitigate further ground displacements.

Alert Level – A maximum value of 15mm relative to the baseline readings is suggested for this project. If this level is reached, the Contractor shall cease construction operations and to execute pre-planned measures to secure the site, to mitigate further movements and to assure safety of public and maintain traffic.

Review of Contractor's Proposed Method

MTO, the Proponent's prime consultant and Foundation Engineer should review the Contractor's proposed method of construction. The proposed method should include a description of the potential loss of ground, and calculation of the maximum settlement in relation to the Contractor's procedure and equipment, alternative/remedial measures when review level of measurement is reached; and contingency/remedial measures when alert level of measurement is reached.

Contractor's Responsibility For Restoration and Warranty Provision

In addition to the monitoring program to assess the adequacy of the construction method to control potential ground movements and groundwater, the Contractor is responsible for reinstatement (such as surface paving) should movements or other surface distress occur, and provide a reasonable warranty period acceptable to MTO. Remedial measures shall be approved by MTO; however, MTO maintains the right to perform the maintenance at the proponent's expense.

Construction Monitoring

The Proponent shall retain a qualified Geotechnical Consultant to supervise the installation of surface settlement points on site and to provide direction, technical input and field inspection on this project.