



**FOUNDATION INVESTIGATION
AND DESIGN REPORT
PETERSON CREEK
CULVERT REPLACEMENT
HIGHWAY 636
TOWNSHIP OF CLUTE
AGREEMENT No.: 5010-E-0006
GWP: 5481-09-00
WP: 5481-09-01
GEOCRES NO.: 42H-44**

March 2012

DST Reference No. GS-TB-012144

Prepared for:

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DST CONSULTING ENGINEERS INC.

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PART 1: FACTUAL INFORMATION

1. INTRODUCTION

DST Consulting Engineers Inc. has been subcontracted by Genivar who was retained by the Ministry of Transportation (MTO), Northeastern Region, to conduct a geotechnical investigation for the replacement of the Peterson Creek culvert on Highway 636. This work was carried out under Agreement No.: 5010-E-0006, Detailed Design for the Replacement/Rehabilitation of Various Culverts.

This report addresses the field investigation, laboratory test program, factual report on file findings (Part 1) and recommendations for design and construction for the proposed culvert replacement (Part 2).

2. SITE DESCRIPTION

The site is located on Highway 636, approximately 3.7 km north of the Highway 11 and Highway 636 intersection, Township of Clute, Cochrane Area. The structural site number is 39E-257.

Existing structure at this location is a 2.85 x 32.95 m Structural Plate Corrugated Steel Pipe (SPCSP) culvert built in 1983 with a depth of soil cover of approximately 4 m. The culvert was identified to be in poor condition with significant sagging and reverse curvature. Supporting column has been installed approximately along the center line but deformation between points of support is evident. It is understood that the existing culvert will be replaced by a 3.0 x 2.4 x 36.0 m pre-cast box structure and replacement will be performed with staged construction method involving installation of roadway protection and temporary widening of the embankment.

The embankment slopes at this location are approximately 2H:1V except the slopes, closer to the existing culvert on the west side, where they were identified to be approximately 1.5H:1V. Both sides of the embankment were sparsely vegetated granular material (Figures 2.5 and 2.6). The photographs shown in Figures 2.1 to 2.4 were taken by MTO and photographs shown in Figures 2.5 to 2.8 were taken by DST during a site visit September 23rd, 2010.

Geological information is available from published *Ontario Geological Survey Map #5036* by the *Ontario Ministry of Natural Resources* for the Smooth Rock area, District of Cochrane. The map indicates that the local area landform is identified as till, clay ground moraine with bedrock plains. The topography in the area is mainly low local relief; plains with mixed wet and dry drainage conditions in the area.



Figure 2.1 Culvert inlet (facing east)



Figure 2.2 Culvert outlet (facing northwest)



Figure 2.3 Culvert supports

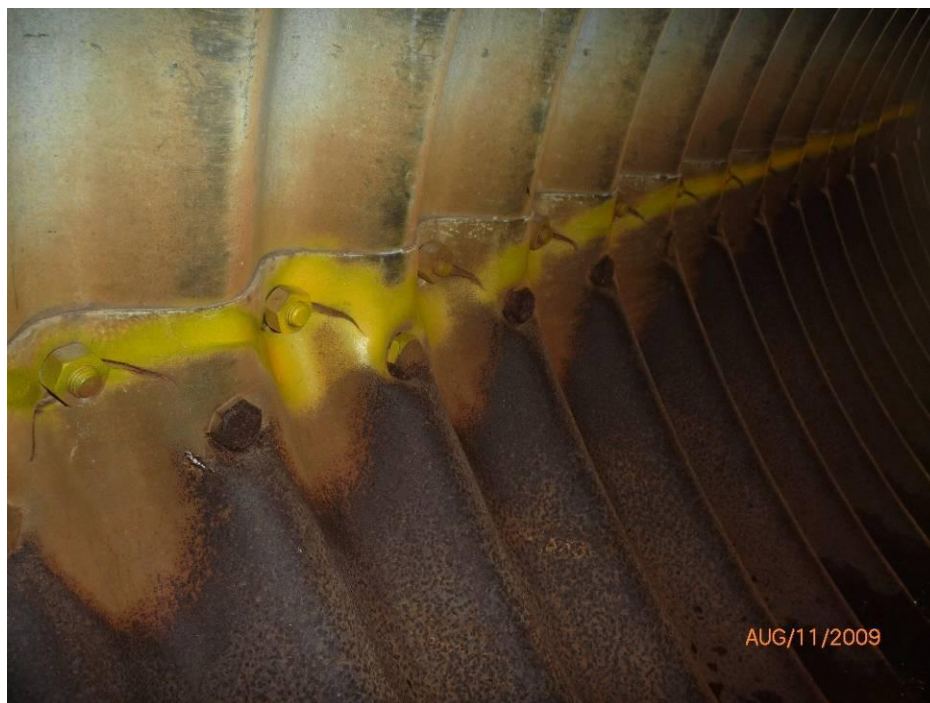


Figure 2.4 Failure at culvert connections



Figure 2.5 Vegetation at culvert inlet (facing west)



Figure 2.6 Vegetation at culvert outlet (facing east)



Figure 2.7 Facing south from culvert



Figure 2.8 Facing north from culvert

3. INVESTIGATION PROCEDURES AND LABORATORY TESTING

Site work was carried out between January 31st, 2011 and February 5th, 2011 utilizing a CME 750 drill rig that was operated by DST personnel. A total of four (4) boreholes were advanced for the purpose of foundation design at this site, two (2) using diamond drilling techniques and two (2) using hand augers.

Two boreholes were advanced through the road structure at Station 10+004 offset 2.5 m right and at Station 9+997 offset 2.5 m left. Two boreholes were advanced at beyond the toe of slope near the existing culvert inlet and outlet. The minimum number of boreholes, and depths and locations of boreholes were chosen according to the given specification in Request for Quotation (RFQ) by MTO. Borehole locations and stratigraphic sections are shown on the Borehole Location Plans, (Drawings 1 to 4).

The borehole locations are referenced to the MTO Station numbering system as indicated in the RFQ. The centreline of the existing culvert was assumed as Station 10+000 as shown in drawing 1. The ground surface elevations at the borehole locations were surveyed by DST personnel. At approximately Station 9+960 at 12 m left a benchmark with elevation of 98.5 m was placed in a guard rail post as shown in Drawing 1. Elevations are correlated to surveyed elevations provided by Genivar. Table 3.1 summarizes the detail of borehole locations and depths.

Table 3.1 Detail of borehole locations

Borehole ID	Station	Elevation (m)	Depth (m)	Offset (m)
BH1	10+004	97.2	20.3	2.5 Rt
BH2	9+997	97.3	15.6	2.5 Lt
BH3	10+015	91.1	1.2	25 Rt
BH4	9+997	92.2	1.0	14 Lt

The fieldwork was supervised on a full-time basis by DST personnel who located the boreholes in the field, performed sampling and in-situ testing and logged the boreholes. Standard Penetration Testing (SPT) and Filed Vane Shear Tests were performed in each borehole. The soil samples collected during drilling were identified in the field, placed in labelled containers and transported to DST's laboratory in Thunder Bay for further analyses.

Classification and index tests were subsequently performed in the laboratory on samples collected from the boreholes to aid in the selection of engineering properties. Laboratory tests included moisture contents, particle size analyses and Atterberg limits including plastic limit and liquid limit. A total of twenty nine (29) moisture contents, six (6) sieve analyses, four (4) particle size analyses and seven (7) Atterberg limit tests have been carried out for this assignment. Laboratory test results are presented in the Boreholes Logs (Enclosures 1 to 4), and Plots (Enclosures 5 to 9). Fines contents obtained from sieve analysis, completed on base and subbase materials.

4. DESCRIPTION OF SUBSURFACE CONDITIONS

The subsurface conditions are presented based on the information obtained during field and laboratory testing.

The generalized stratigraphy of the existing embankment, based on the conditions encountered in boreholes, consists of surface treatment overlying a sand with some crushed gravel and clay fill that is underlain by a sand fill. This fill is then underlain silty clay with lenses of sand found in some locations.

Table 4.1 Depths and elevations of auger refusals

Borehole ID	Depth of auger refusal (m)	Elevation of auger refusal (m)
BH3	1.2	89.9
BH4	1.0	91.2

4.1 Surface Treatment

Surface treatment was encountered in Boreholes 1 and 2 with a thickness of approximately 50 mm.

4.2 Embankment Fill

Thickness of the fill is between approximately 7.4 and 7.5 m at this location. Within the sand fill, cobbles and rock fill were encountered during the drilling process. Within the clay fill wood debris was encountered during the drill process. Grain size distributions of the fill material are reported in borehole logs (Enclosures 1 to 4) and plots (Enclosures 5 through 6).

A pavement structure beneath the surface treatment of sand and some crushed gravel and silt was identified in Boreholes 1 and 2 from 50 mm below surface to depths up to 0.18 m; this corresponds to maximum and minimum upper and lower boundary elevations of approximately 97.3 and 97.0 m respectively. Gradation analyses conducted on a sample from Borehole 1 indicates gravel, sand, and fines contents of approximately 18%, 65% and 17% respectively. This material does not classify as Granular A meeting SSP 110S13 requirements. The moisture content of samples was 8%.

Beneath this sand with some crushed gravel and silt a fill of predominantly loose to compact

sand materials was encountered at Boreholes 1 and 2 from 0.16 m below surface to depths up to 6.0 m; this corresponds to maximum and minimum upper and lower boundary elevations of approximately 97.1 and 91.3 m respectively. In Boreholes 1 and 2 rock fill and cobbles were encountered within the sand fill from approximately 0.6 to 2.3 m and 4.6 and 6.0 m below surface; this corresponds to maximum and minimum upper and lower boundary elevations of approximately 96.7 and 91.3 m respectively. Gradation analyses conducted on samples from Boreholes 1 and 2 indicate gravel, sand, and fines contents of approximately 1 to 17%, 70 to 98% and 2 to 21% respectively. Some of this material classifies as Granular B, Type I meeting SSP 110S13 requirements. The moisture content of samples was between 3 and 22%.

Beneath this loose to compact sand a silty clay fill was encountered in Boreholes 1 and 2 between 5.3 and 7.5 m below surface; this corresponds to maximum and minimum upper and lower boundary elevations of approximately 92.0 and 89.8 m respectively. The moisture content of samples was between 16 and 39%.

Beneath this silty clay fill a compact sand material was encountered between depths of 6.8 and 7.5 m below surface in Boreholes 1; this corresponds to maximum and minimum upper and lower boundary elevations of approximately 90.5 and 89.7 m respectively. The thickness of the stratum was determined to be approximately 0.7 m in thickness. Gradation analyses conducted on a sample from Borehole 1 indicates gravel, sand, and fines contents of approximately 25%, 69% and 6% respectively. The moisture content of a sample was 11%.

4.3 Organics

Organic material, decomposed wood, was encountered mixed with the embankment fill materials in Boreholes 1 and 2 between depths of approximately 4.9 and 6.8 m; this corresponds to maximum and minimum upper and lower boundary elevations of approximately 92.3 and 90.5 m respectively. The moisture content of a sample was 167%.

4.4 Topsoil

Topsoil with variable gradations and organics was encountered at surface in Boreholes 3 and 4 at depths between surface and 60 mm; this corresponds to maximum and minimum upper and lower boundary elevations of approximately 91.05 and 90.9 m in Borehole 3 and 92.15 and 91.9 m in Borehole 4 respectively.

4.5 Clay

Silty clay was encountered in Boreholes 1 through 4. It was encountered at depths from 5.3 to 20.3 m; this corresponds to maximum upper boundary elevations of approximately 89.8 m and 92.1 m in Boreholes 1 and 2 and Boreholes 3 and 4 respectively. The thickness of this stratum is not defined in Boreholes 1 through 4 as borehole terminus was reached within the stratum. Atterberg limit tests carried out on samples from Boreholes 1, 2 and 3 indicate this clay varies from low to high plasticity with liquid limits and plasticity indexes from 29 to 65 and 14 to 40 respectively. In-situ field vane tests taken in Boreholes 1 and 2 indicate undrained shear strengths between 55 and 220 kPa with sensitivities ranging from 3 to 4. The moisture contents of samples range from 16% to 55%.

4.6 Groundwater

The groundwater table was identified below the ground surface during the field investigation and through visual identification of soil samples. The estimated depth of groundwater level below the ground surface elevation is given in Table 4.2. The water levels at the culvert inlet and outlet were at elevations of approximately 92.0 m and 91.1 m respectively during the field investigation. The groundwater levels and water level at the culvert can be expected to vary with season and precipitation events.

Table 4.2 Depth of water table at boreholes

Borehole ID	Borehole elevation (m)	Water table elevation (m)	Depth of water table below the ground surface (m)
BH1	97.2	92.1	5.1
BH2	97.3	91.5	5.8
BH3	91.1	90.1	1.0
BH4	92.2	92.1	0.1

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PART 2: ENGINEERING DISCUSSIONS AND RECOMMENDATIONS

5. PROJECT DESCRIPTION

DST Consulting Engineers Inc. has been subcontracted by Genivar who was retained by the Ministry of Transportation (MTO), Northeastern Region, to conduct a geotechnical investigation for the replacement of the Peterson Creek culvert on Highway 636. This work was carried out under Agreement No.: 5010-E-0006, Detailed Design for the Replacement/Rehabilitation of Various Culverts.

This proposed culvert is to be replaced by a pre-cast box structure (3.0 x 2.4 x 37.5 m). The proposed culvert invert elevation varies from 90.4 to 90.6 m. A staged construction method involving installation of roadway protection and widening the embankment is the preferred replacement approach.

The generalized stratigraphy of the existing embankment, based on the conditions encountered in boreholes, consists of surface treatment overlying a fill of sand with some crushed gravel and silt that is underlain by sand fill and clay fill. This fill is then underlain in areas by silty clay with some sand lenses found at some locations. The water level in the creek at the culvert was at an elevation of approximately 92.1 m at the time of the investigation.

This section presents interpretation of the geotechnical data presented in the factual report and presents geotechnical design recommendations and construction concerns for the proposed culvert replacement.

5.1 Precast Concrete Box Culvert

For this culvert replacement, a precast concrete box culvert is to be used. Open cut excavation has been considered for the replacement of the structure.

The design of the culvert must be in accordance with the Canadian Highway Bridge Design

Code CAN/CSA-S6-06 and all relevant Ministry of Transportation specification and guidelines.

5.1.1 Earth Excavation

An open cut operation along the proposed culvert alignment is proposed by MTO for the culvert replacement. This method of construction may result in traffic disturbances and may require temporary surface water ditch diversion and temporary support for traffic. It can more readily accommodate excavation of large boulders with this method, if encountered during excavation. As a minimum, the procedures should be in accordance with OPSS 902 "Construction Specifications for Excavating and Backfilling-Structures". Where temporary protection systems are required they shall be constructed in accordance with OPSS 539 "Construction Specification for Temporary Protection Systems" and Section 5.1.6 Roadway Protection.

If organic materials are encountered during excavation, the excavations to remove these organics and wood should be completed in accordance with OPSD 203.040. It is anticipated that the existing groundwater table will be above the invert level. Excavation below the water table can be undertaken by either dewatering of the excavation or in the wet without lowering the water table. If excavation is completed in the wet, any sub-excavated materials can be replaced with 19 mm Type I or II clear stone as defined in OPSS 1004.05.02. If fine materials are present beneath the clear stone a non-woven geotextile (OPSS 1860.07.05.01 Class II) with the filtration opening size (FOS) less than 135 µm may be required for separation. Proof rolling of clear stone is required prior to the placement of subsequent materials.

5.1.2 Staged Construction

Staged construction has been identified by prime consultant, Genivar, as the preferred approach to maintain traffic during the construction of the culvert at this site. The proposed staged construction includes two (2) stages as given in Genivar Staging Drawings in Appendix B. Slope stability analyses for the proposed slope geometries have been conducted and are presented in Section 5.1.4 Embankment Design.

Stage 1 is a temporary lane diversion which involves temporary detour of traffic to the southbound lane of 6.26 m width with existing side slopes of approximately 2.5H:1V and as steep as 1.5H:1V in the granular and rock fill materials respectively as well as the installation of level II roadway protection. Excavation adjacent the roadway protection is anticipated to an elevation of approximately 89.6 m to allow for placement of bedding materials. Use of temporary concrete

barriers will be required.

Stage 2 is a temporary lane diversion which involves temporary detour of traffic to the northbound lane of 6.26 m width with temporary side slopes of 3H:1V and 1.25H:1V in the granular and rock fill materials respectively as well as the installation of level II roadway protection. Excavation adjacent the roadway protection is anticipated to an elevation of approximately 89.9 m to allow for placement of bedding materials. Use of temporary concrete barriers will be required.

The final embankment foreslopes should be reinstated as presented in Section 5.1.12 Embankment Foreslopes.

5.1.3 Foundation Design

The culvert will be located approximately at the same elevation and location as the existing culvert. As the proposed culvert is not expected to be heavily loaded, a shallow foundation is considered suitable for this site. As the cross sectional area of the box culvert structure will be slightly larger than the existing structure, the overall effect on the culvert foundation soils will be a small decrease in stress at the base of the culvert.

The geotechnical resistance was estimated for the ultimate limit state (ULS) and serviceability limit state (SLS) for a maximum settlement of 25 mm. The resistance at ULS was calculated by applying load resistance factor of 0.5 according to the Bridge Design Code (CHBDC) CAN/CSA-S6-06 section 6.6.3.6, Table 6.1. The geotechnical resistance was estimated assuming a strip footing consisting of a width equal to the width of the culvert (3.0 m) and a depth of the culvert base equal to 0 m, which is a temporary worst condition prior to backfill that will be encountered during construction. Settlement of the structure can be considered negligible due to the marginal increase in net loading. While ULS is not relevant at final condition due to excessive soil cover SLS is not relevant for temporary condition. Therefore SLS reported here are for final condition.

Table 5.1 Geotechnical resistances and reactions

Footing Size	Ultimate bearing capacity (kPa)	Factored Resistance at ULS (kPa)	Resistance at SLS (kPa)
B = 3.5 m	250	125	50

The width of the sub-excavation should be twice the width of the culvert and where unsuitable or unstable soils are encountered, the foundation soils must be removed to a firm or hard

soils and replaced to the foundation grade. If sub-excavation for frost effects is carried out in the dry (with adequate dewatering controls), the material can be replaced with Granular A material meeting SSP 110S13 specifications and compacted to a minimum of 95 % of standard Proctor maximum dry density in accordance with OPSS 501. If sub-excavation for frost effects is carried out in the wet (water is maintained at or above adjacent groundwater table) All foundation preparation should be completed as required by OPSS 422, as specified in the contract documents and as indicated in Section 5.1.7 Bedding.

5.1.4 Embankment Design

Slope stability analyses were carried out with limit equilibrium methods using Geoslope version 2004 software applying Morgenstern and Price methods. Targeted factor of safety for slope stability analyses was 1.3 for permanent stability analyses. Slope stability analyses were performed under the following slope conditions with an embankment height of up to 7.0 m:

- Stage 1 temporary embankment with minimum 2H:1V and 1.5H:1V in granular and rock fill foreslopes over existing southbound lane,
- Stage 2 temporary embankment with minimum 2H:1V and 1.5H:1V in granular and rock fill foreslopes over reinstated northbound lane,
- Reinstated southbound lane embankment after culvert replacement, 2.0H:1V rock fill foreslopes,
- Reinstated northbound lane embankment after culvert replacement, 2.0H:1V rock fill foreslopes.

Results for both undrained and drained conditions of the evaluated slope configurations and are presented in Table 5.2 despite drained condition may not relevant for some temporary condition.

Table 5.2 Summary of stability analyses

Slope Condition	Foreslope Gradient	Drained or Undrained Analyses	Factor of Safety
Stage 1: Temporary embankment over existing southbound lane, excavation of northbound lane	2H : 1V	Drained	1.3
		Undrained	1.5
Stage 2: Temporary embankment over reinstated northbound lane excavation of southbound lane	1.5H : 1V	Drained	1.3
		Undrained	>1.5
Embankment after culvert replacement, southbound lane	2.0H : 1V	Drained	1.3
		Undrained	>1.5
Embankment after culvert replacement, northbound lane	2.0H : 1V	Drained	1.3
		Undrained	>1.5

This analyses considered the soil parameters as defined in Table 5.3 and a water table at 5.0 m below the top of embankment in reinstated and temporary embankment conditions.

Excavation of temporary side slopes above the water table that do not support traffic should not be steeper than 1.0H:1.0V, although, flatter slopes may be required depending on construction methods. Temporary rock fill slopes above the water table supporting traffic during the construction stages should not be steeper than 1.5H:1V. Temporary granular slopes above the water table supporting traffic during the construction stages should not be steeper than 2H:1V. Design of temporary slopes below the water table will depend on the dewatering method. Embankment foreslopes should be reinstated as indicated in Section 5.1.12 Embankment Foreslopes.

The trench width must be sufficient to permit proper use of compaction equipment suited for the material to be compacted, to reach the degree of compaction required, and to accommodate within the space available as per OPSS 501, "Construction Specification for Compaction".

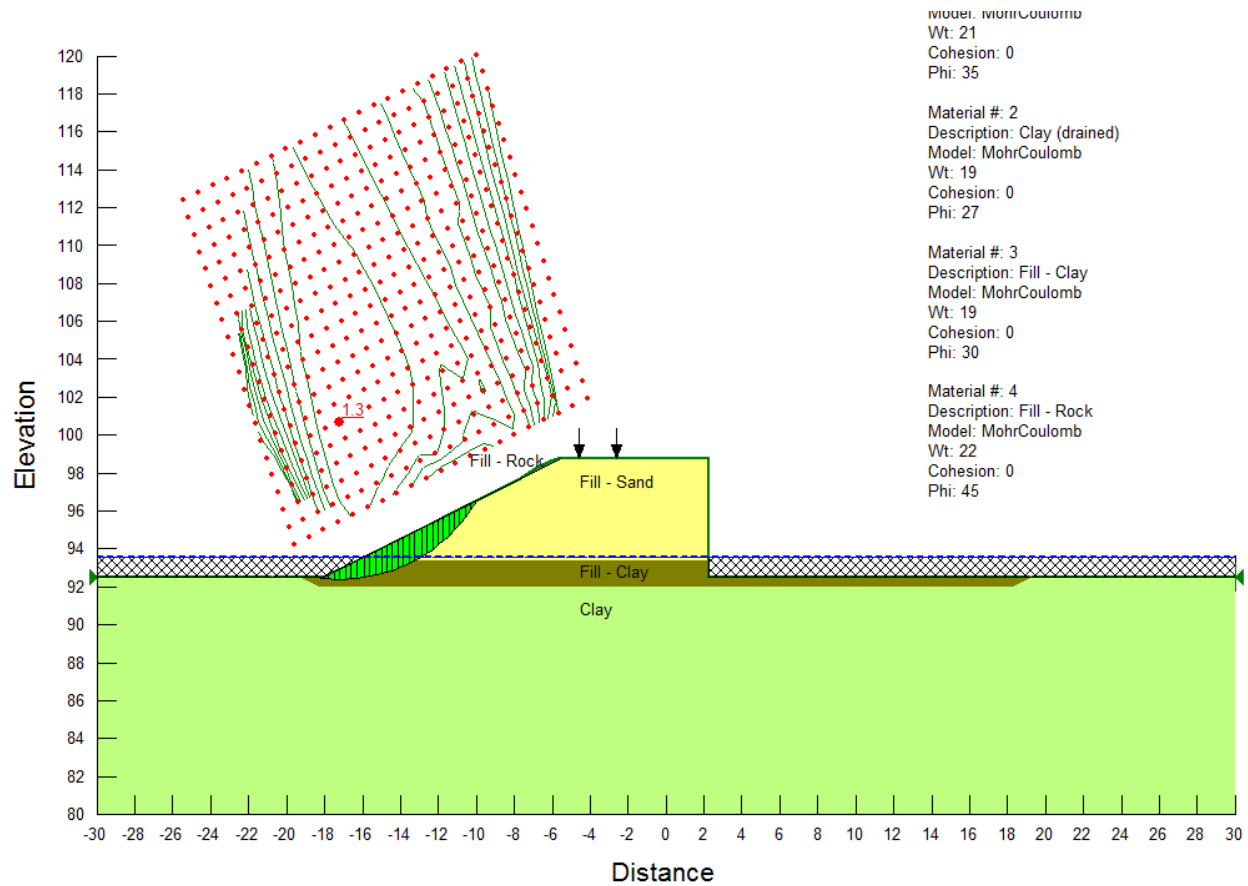


Figure 5.1 Slope stability analysis Stage 1 temporary embankment with minimum 2H:1V and 1.5H:1V in granular and rock fill foreslopes over existing southbound lane under drained condition

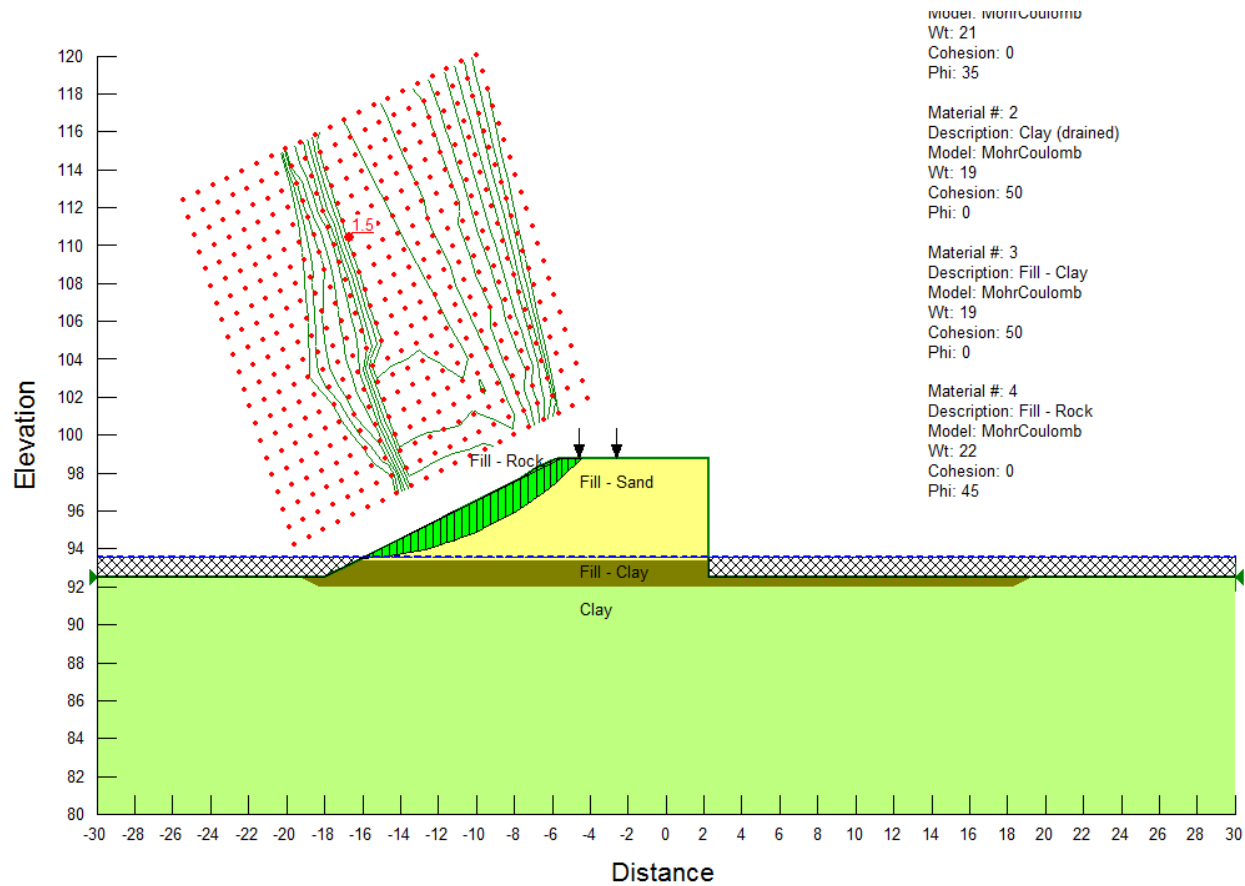


Figure 5.2 Slope stability analysis Stage 1 temporary embankment with minimum 2H:1V and 1.5H:1V in granular and rock fill foreslopes over existing southbound lane under undrained condition

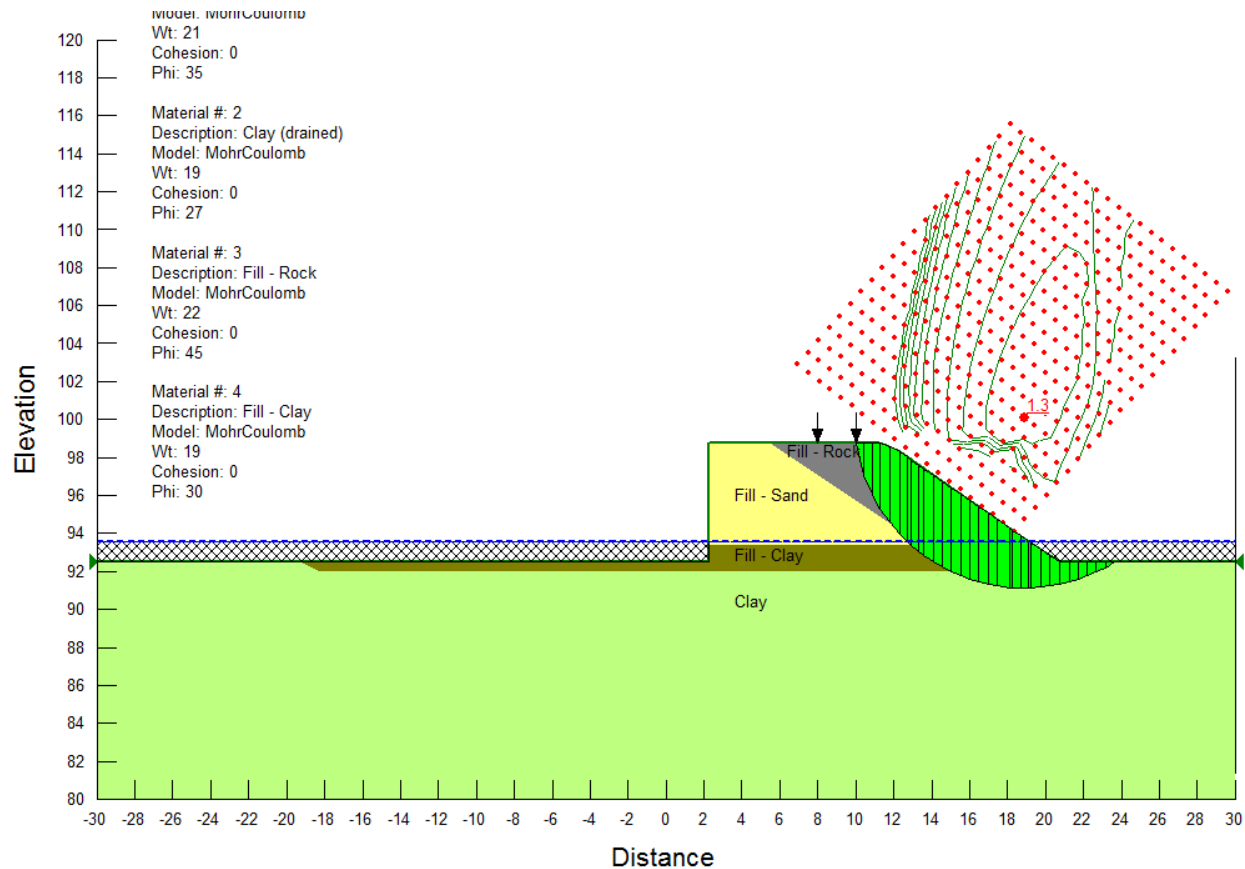


Figure 5.3 Slope stability analysis Stage 2 temporary embankment with minimum 2H:1V and 1.5H:1V in granular and rock fill foreslopes over reinstated northbound lane under drained condition

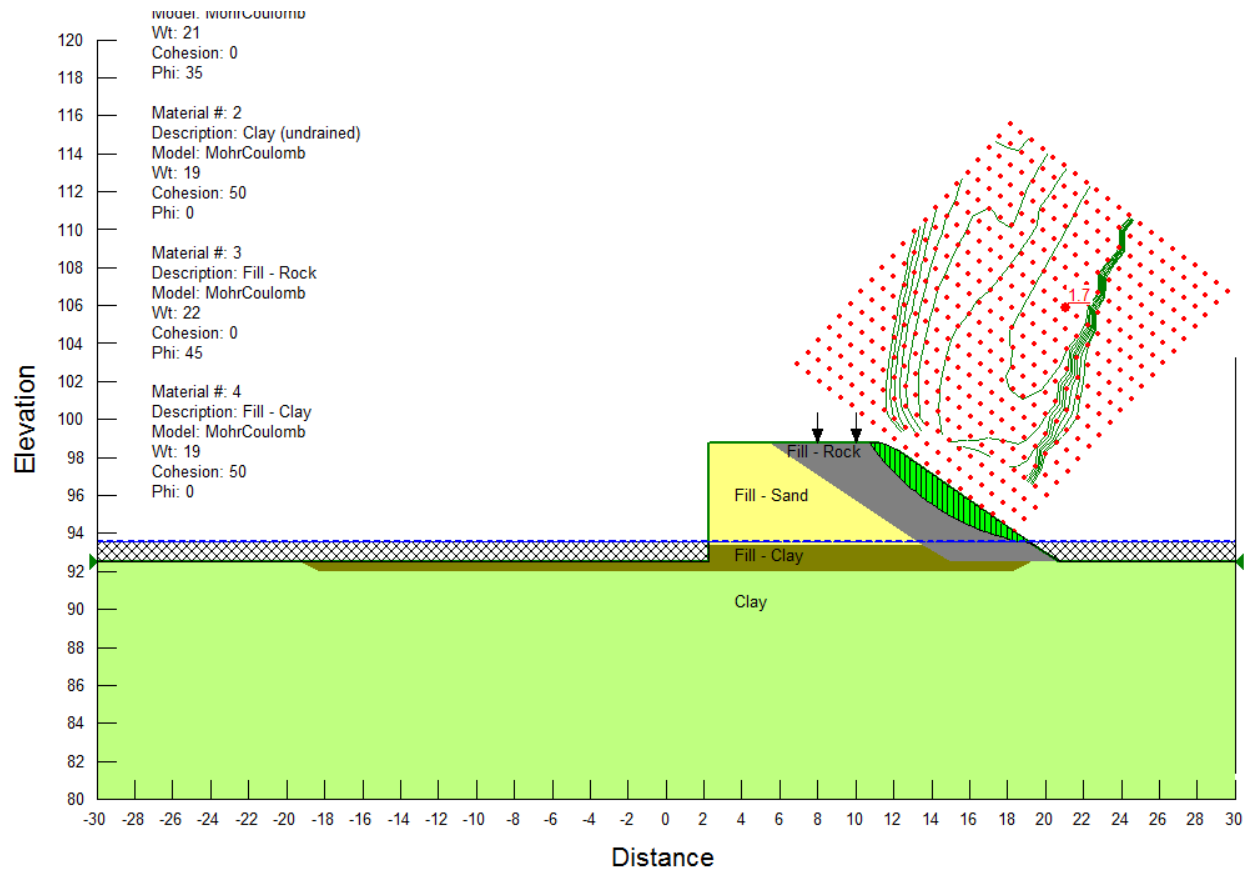
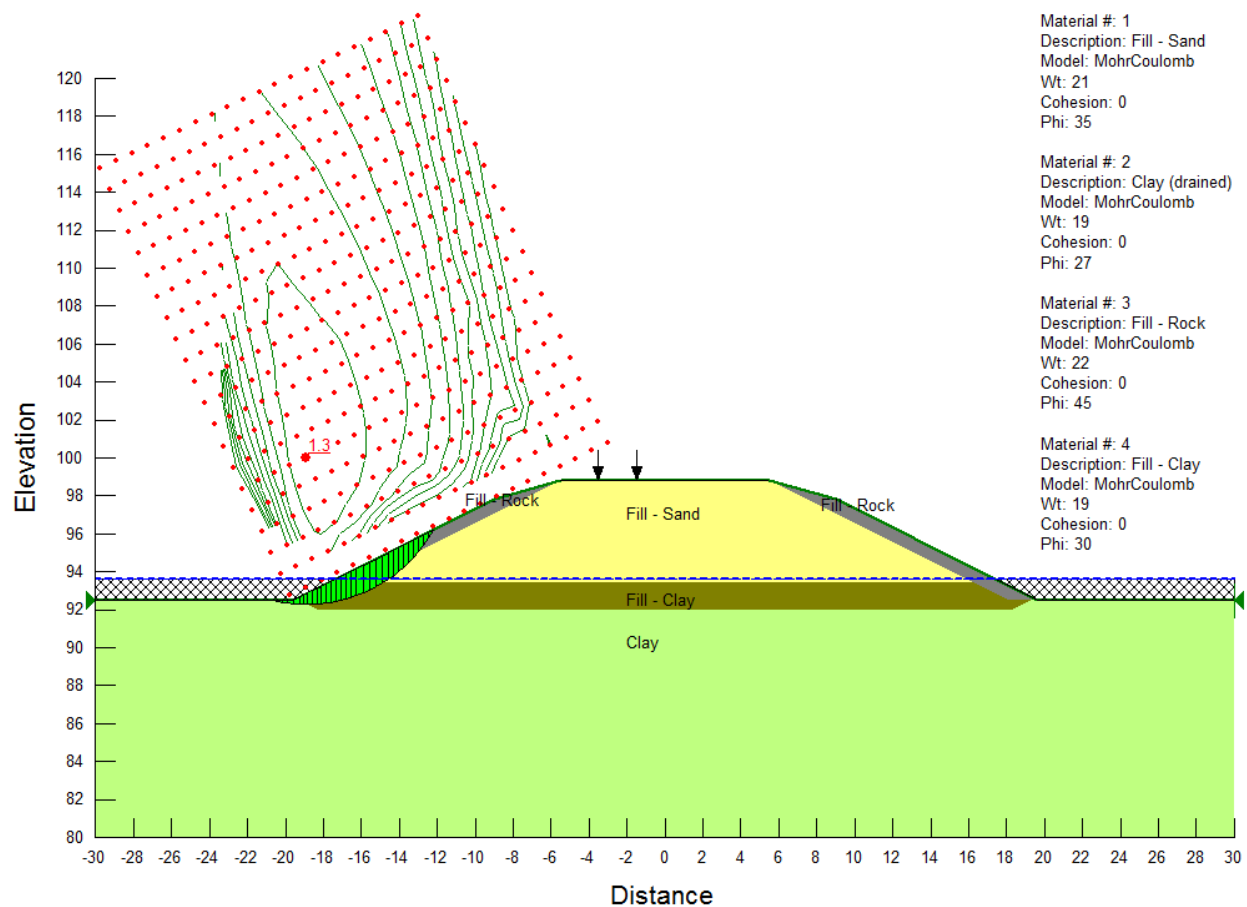


Figure 5.4 Slope stability analysis Stage 2 temporary embankment with minimum 2H:1V and 1.5H:1V in granular and rock fill foreslopes over reinstated northbound lane under undrained condition



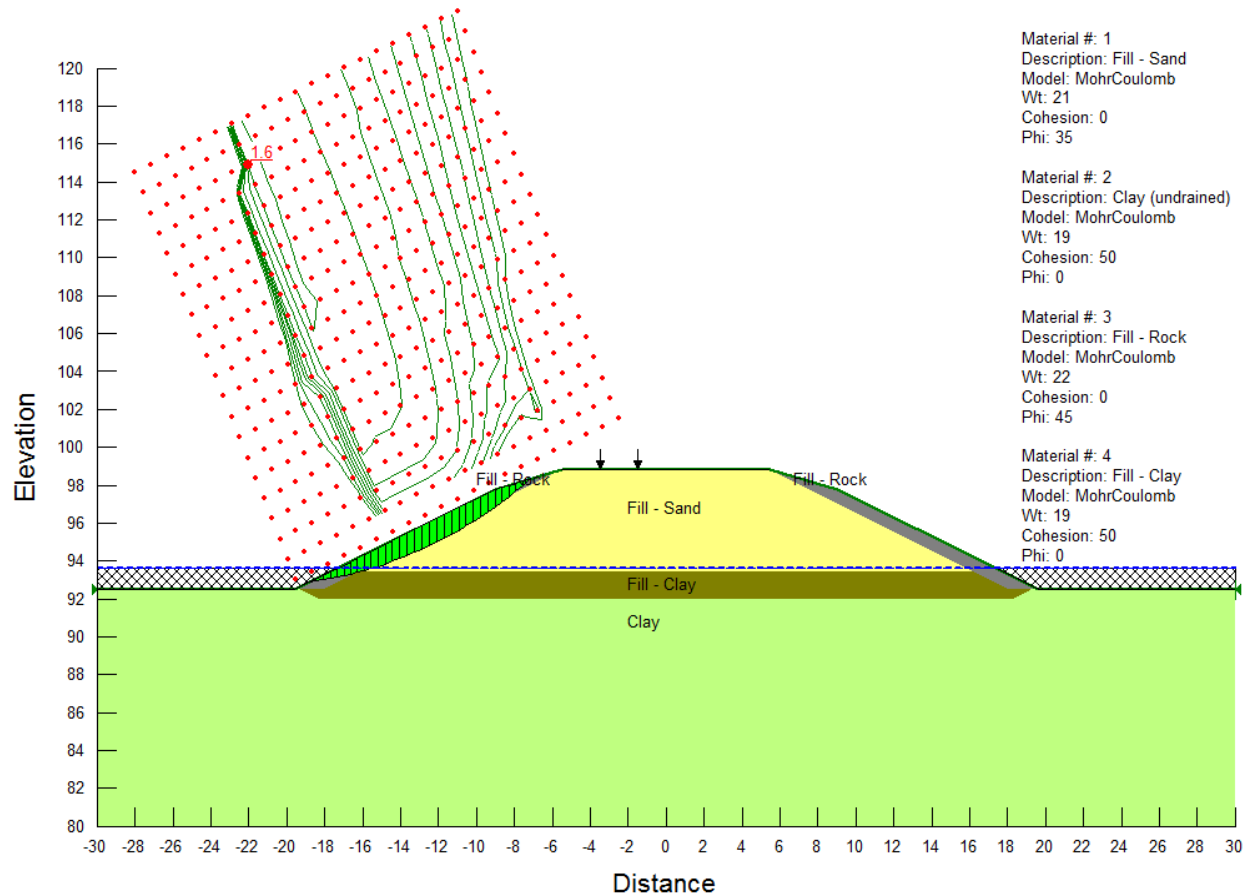


Figure 5.6 Slope stability analysis reinstated southbound lane embankment after culvert replacement, 2.0H:1V rock fill foreslopes under undrained condition

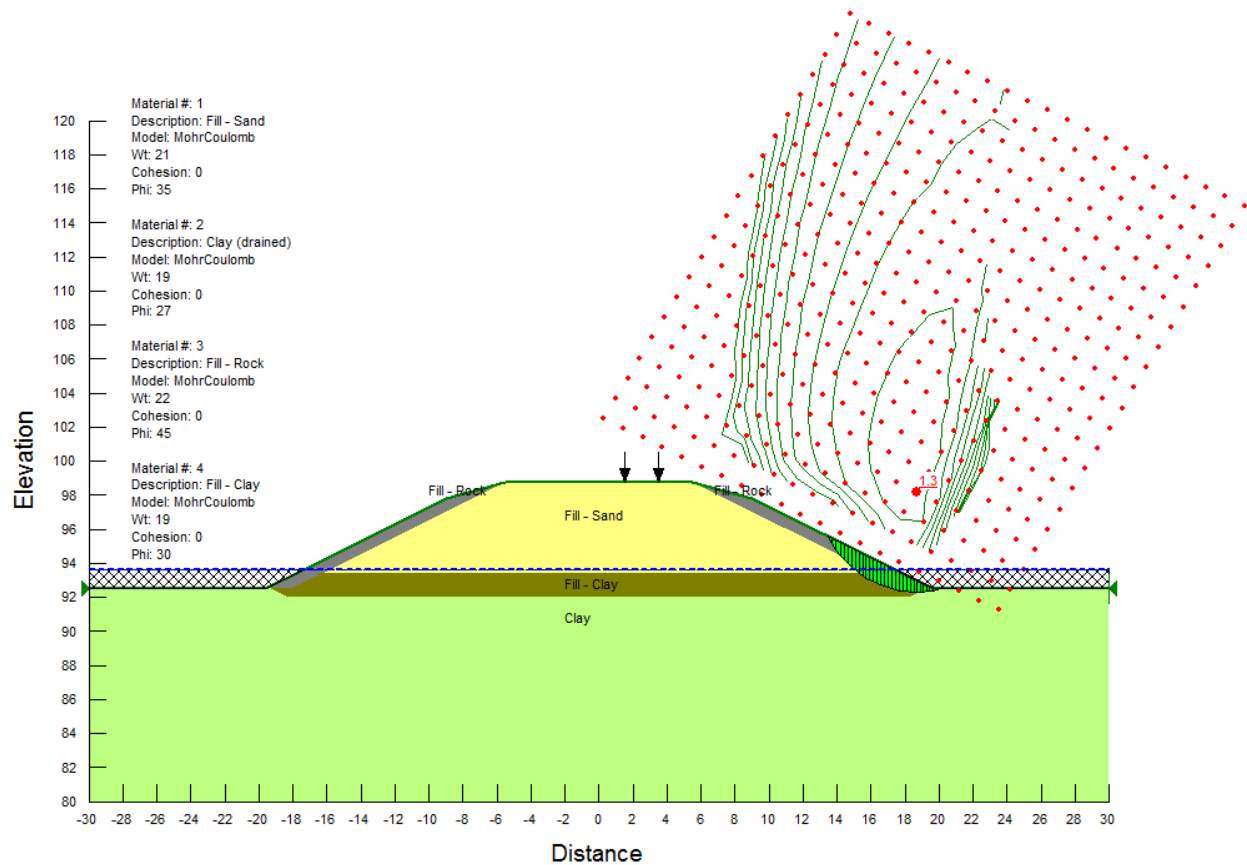


Figure 5.7 Slope stability analysis reinstated northbound lane embankment after culvert replacement, 2.0H:1V rock fill foreslopes under drained condition

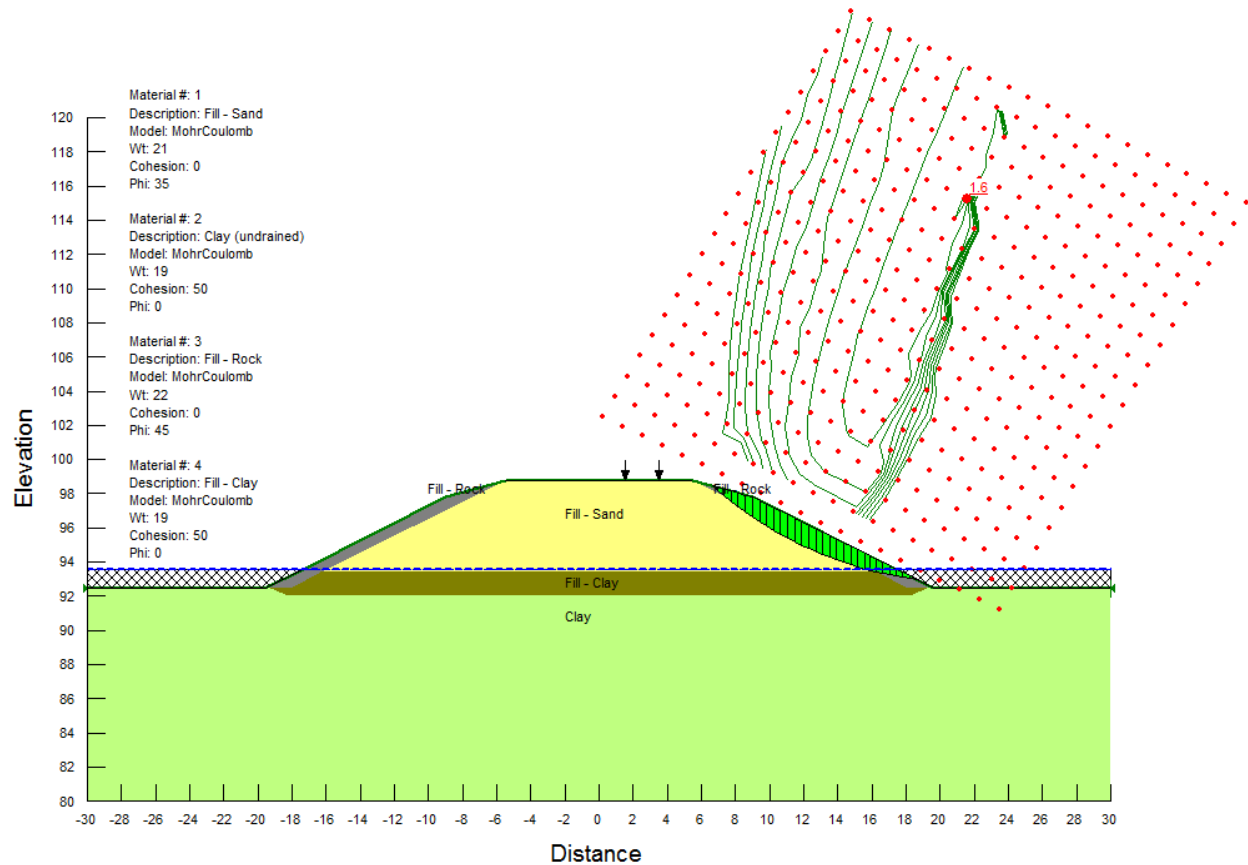


Figure 5.8 Slope stability analysis reinstated northbound lane embankment after culvert replacement, 2.0H:1V rock fill foreslopes under undrained condition

5.1.5 Lateral and Sliding Resistances

The analysis of horizontal and vertical effects of earth pressures on the culvert can be performed considering soil parameters given in Table 5.3 and assuming linearly variation of stress change with the depth as described in Section 7.8.5.3.2 in CHBDC. Temporary shoring may be designed using the typical soil parameters given in Table 5.3, but the designer/contractor should verify the appropriate soil parameters for the designs of specific shoring system.

Concrete toe walls are proposed by the prime consultant to be constructed at the inlet and outlet of the culvert and should be constructed in accordance with OPSD 3120.100.

It is recommended that all excavations be either adequately sloped or securely shored and braced to prevent earth caving and to provide a safe and stable work area. The design should incorporate the effects of hydrostatic pressure, traffic surcharge and retained sloping earth conditions in the bracing design.

Resistance to lateral forces/sliding resistance between the base slab for the replacement of culvert and subgrade should be calculated in accordance with section 6.7.5 of the CHBDC.

The coefficients for lateral earth pressure can be calculated using equations provided in Table 5.4. Where no significant earth movements are expected, the coefficient K_o should be used.

Table 5.3 Typical soil parameters for earth loads

Soil type	Unit weight (kN/m ³)	Internal friction angle (Deg)	Interface friction angle, δ (Deg)	Intact undrained shear strength (kPa)
Rock Fill	22	45	-	-
Granular A	21	35	17	-
Granular B	21	35	17	-
Sand Fill	21	35	17	-
Clay Fill	19	30	13	50
Clay	19	27	13	50

Table 5.4 Lateral earth pressure coefficients

Earth Pressure Coefficient	Equation*
Active Earth Pressure (K_a)	$\left(\frac{1 - \sin\phi}{1 + \sin\phi} \right)$
Passive Earth Pressure (K_p)	$\left(\frac{1 + \sin\phi}{1 - \sin\phi} \right)$
At rest (K_0)	$(1 - \sin\phi)$

* ϕ is an angle of internal friction

5.1.6 Roadway Protection

Roadway protection for this project should be constructed in accordance with the requirements of the Occupational Health and Safety Act of Ontario (OHSA), O.Reg. 213/91. According to O.Reg. 213/91, s.226, the soils in the area of interest classify as Type 3 and Type 4 if located above and below the water table respectively. Type 3 soils generally are stiff to firm and compact to loose or are previously excavated soil, exhibit signs of surface cracking, exhibit signs of seepage, if it is dry, may run easily into a conical pile and have a low degree of internal strength. Type 4 soils generally are soft to very soft and very loose in consistency, very sensitive and upon disturbance are significantly reduced in natural strength, run easily or flow unless it is completely supported before excavation procedure, have almost no internal strength, are wet or muddy and exerts substantial fluid pressure on its supporting system. In accordance with O. Reg. 213/91, s.227 (3), if an excavation contains more than one type of soil, the soil shall be classified with the highest number as described in section 226. These should be assessed and confirmed in the field as construction progresses.

Since roadway protection is required during the culvert replacement, installation of a cantilevered sheet pile system may be considered to ensure the stability of the bank and is a feasible option. Alternatively, the use of soldier piles with lagging installed as the excavation progresses may also be considered. Soldier piles, properly designed, will be more capable of accommodation the presence of cobbles expected to be encountered within the embankment fill. The design of sheet pile or soldier pile walls may be performed using the typical soil parameters given in Table 5.3, but the designer/contractor should verify the appropriate soil parameters for the designs. Since the embankment is not to be reduced in height, the potential of encountering rock fill, cobbles and boulders is likely. The contractor should be prepared to handle the presence of rock fill

with the selection of adequate driving or vibratory equipment as well as steel thickness.

The construction methodology must be in accordance with OPSS 539 “Construction Specification for Temporary Protection Systems” as well as all Ministry of Transportation, Ministry of Environment, Ministry of Natural Resources and Department of Fisheries and Oceans guidelines, and also the Occupational Health and Safety Act of Ontario. The contractor’s method and equipment must be suitable for the site conditions and materials used.

5.1.7 Bedding

The bedding for the structure should be designed in accordance with the contract documents, Section 7.8.3 of the CHBDC and as specified in OPSS 422 “Construction Specification for Precast Reinforced Concrete Box Culverts and Box Sewers in Open Cut”.

The foundation soils, clay in particular, will be very susceptible to disturbance and weakening as a result of traffic, standing water and frost. Any foundation soils that could be disturbed shall be protected. The bottom of the excavation on which the culvert or granular pad is to rest shall not be disturbed. The bedding placement should commence immediately after the final removal of material to the foundation level has been completed.

The bedding shall be a minimum of 0.5 m thick and extend to a minimum width (half of the width of culvert) beyond all sides of the culvert. The bedding material should consist of Granular A as per Soil Group I in accordance with Table 7.4 of the Canadian Highway Bridge Design Code. The Granular A shall be in accordance to SSP 110S13. The Granular A should be placed in layers not exceeding 200 mm in thickness, loose measurement, and each layer compacted to a minimum of 95 % of standard Proctor maximum dry density in accordance with OPSS 501. The middle one-third of the culvert width of the top bedding layer, having minimum thickness of 75 mm, shall be loosely placed and uncompacted.

If construction is performed without dewatering bedding material should consist of 19 mm Type I or II clear stone as defined in OPSS 1004.05.02. If fine materials are present beneath the clear stone a non-woven geotextile (OPSS 1860.07.05.01 Class II) with the filtration opening size (FOS) less than 135 µm may be required for separation. Proof rolling of clear stone is required prior to the placement of subsequent materials.

As no additional grade raise is anticipated at the culvert location, only marginal changes in

net loading above the culvert replacement is anticipated, settlements should then be considered to be occurring under a recompression condition. Therefore, relative settlements along the culvert flow path can be considered negligible and camber of the bedding is not required.

5.1.8 Sidefill and Overfill

The sidefill and overfill for the structure should be designed in accordance the contract documents, Section 7.8.3 of the CHBDC and as specified in OPSS 422 "Construction Specification for Precast Reinforced Concrete Box Culverts and Box Sewers in Open Cut".

The material used for culvert sidefill should not contain debris, organic matter, frozen materials, or large stones, must meet SSP110S13 Granular A requirements and be compacted to 95% of standard Proctor maximum dry density in accordance with OPSS 501. Soils shall be deposited uniformly on each side of the structure in order to prevent lateral displacement. The minimum width of the sidefill should be at least half of the culvert width in each side.

Overfill should consist of Granular A and should be compacted to a minimum of 90 or 95 % of Standard Proctor Maximum Dry Density (SPMDD) respectively but not greater than the compaction or equivalent stiffness of soils in the sidefill and bedding zones. All compaction shall be completed in accordance with OPSS 501. The Granular A shall be in accordance to SSP 110S13. Each layer should not exceed 200 mm in thickness, loose measurement. The backfill materials should be separated from the adjacent soil with a non-woven Class II geotextile specified in OPSS 1860.

When a concrete culvert is installed on the undisturbed original ground and fill material is placed around and over the culvert, relative settlements between the fill adjacent to the sides of the culvert and the fill directly over the culvert generates downward frictional forces on the culvert, also effecting a load transfer. This vertical load on the culvert can be determined by multiplying the weight of earth over the top of the box section by the vertical arching factor, λ_v . Vertical arching factors for Type B1 and B2 box culverts in standard installations can be considered 1.20 and 1.35 respectively as indicated in Section 7.8.4.2.3 of the CHBDC.

$$q = \gamma h b \lambda_v, \text{ where}$$

q = vertical load on the culvert

γ = unit weight of soil

h = thickness of soil above the culvert

b = width of the culvert, and

λ_v = vertical arching factor

However, due to the marginal change in net loading above and directly adjacent the culvert replacement, settlements should be considered to be occurring under a recompression condition. Therefore, relative settlements between the fill adjacent the sides of the culvert and the fill directly over the culvert can be considered negligible which results in no or little downdrag force.

5.1.9 Channel Diversion and Dewatering

The culvert shall be replaced by diverting the creek through a temporary bypass adjacent to the existing culvert. It is important to ensure that a flood in the bypass does not cause damage to the partly constructed permanent works, to the temporary works or to plant. Floods can occur quickly and can cause significant financial consequences if adequate containment strategies are not present.

If the creek has comparatively a small amount of flow that may depend on the season, it may be feasible for the creek flow to be directed by staging construction. In order to prevent back up of water from upstream and downstream, a dyke made of sand bags has sometimes been used as a hydraulic barrier. However, a sheet pile vertical cut-off wall will provide better control of both surface and groundwater. An adequately designed and properly installed sump and pump system will be sufficient to dewater due to low permeable nature of underlying soils and stabilize the excavation without risk of soil disturbance. It should be noted that depending on the season, depth of excavation and amount of water flow through the creek may vary. The contractor should be prepared to tackle this situation. The contractor should be alerted of the high water table and surface water, for example through a non standard special provision (NSSP).

Where dewatering is performed, all dewatering operations should be completed in accordance with OPSS 517 "Construction Specification for Dewatering of Pipeline, Utility, and Associated Structure Excavation". If construction is to be completed in the dry a continuous dewatering operation must be provided to keep the excavation stable and free of water. The excavation must be monitored to confirm this. The dewatering system must be maintained and the surrounding area monitored for impacts to items such as, but not limited to, settlement and

groundwater usage. The control of water from the dewatering operation should be accordance with OPSS 518 “Construction Specification for Control of Water from Dewatering Operations”.

Water shall be disposed of so as not to be injurious to public health or safety, property, the environment, fisheries, or any part of the work completed or under construction. Dewatering operations shall be directed to a sediment control device or natural attenuation area prior to discharge to watercourses. If a natural attenuation area is used, a minimum 15 m setback shall be maintained from the receiving watercourse. When water is discharged to a watercourse, the water discharged shall be done in a manner that does not cause erosion or other damage to adjacent lands.

When required, a permit issued by the Ministry of the Environment (MOE) for taking water from a groundwater source shall be obtained.

5.1.10 Erosion Control

Erosion control is essential at inlet and outlet for the successful performance of a culvert. Generally, rip-rap is used to avoid the erosion at inlet and outlet of the culvert. The rip-rap slows down the flow close to the channel bed and prevents culvert failure by the undermining.

To prevent erosion of the surrounding soils at the inlet, rip-rap Treatment shall be applied accordance with OPSD 810.020 “Rip-Rap Treatment for Ditch Inlets” and OPSS 511 and SP511S01 “Construction Specification for Rip-Rap, Rock Protection, and Granular Sheeting”.

The outlet shall be rip-rapped to prevent erosion of the surrounding soils accordance with OPSD 810.010 “Rip-Rap treatment for Sewer and Culvert Outlets” and OPSS 511 and SP511S01 “Construction Specification for Rip-Rap, Rock Protection, and Granular Sheeting”.

To prevent undermining of the bedding, cutoff walls shall be installed along the entrance and exit end bottom sides of culvert. Cutoff wall should be designed based on velocity of the water flow and the type of soil underneath.

Considering the replacement of Granular A material underneath and in front of the inlet and replacement of cover material with clear stone or granular material, a clay seal should be considered to minimize underflow. A blanket clay seal should be at minimum 300 mm thick and extend 2 m beyond the fill materials. Clay seals should be constructed in accordance with OPSS 422 and have material properties as specified in OPSS 1205. Alternatively, a geosynthetic clay liner or an ethylene

propylene diene monomer (EPDM) liner installed to manufacturer's specifications may also be suitable.

The temporary erosion and sedimentation measures during the construction of culvert shall be controlled as described in OPSS 805 "Construction Specification for Temporary Erosion and Sedimentation Control Measures".

5.1.11 Frost Protection

In accordance with OPSD 3090.100 "Foundation Frost Depths for Northern Ontario", the frost penetration at this location is about 2.6 m. The frost susceptible soils shall not be used adjacent to the culvert wall within the depth of frost penetration from the road surface. The soils under the culvert are highly frost susceptible (capable of forming thick ice lenses with the associated pressures and heave).

During winter season, ice may form inside the culvert and a low flow rate may assist the ice formation. It is expected that ice may extend to the culvert invert and frost could therefore extend into the soils below the culverts, possibly as deep as 2.6 m. The frost heave may generate additional stresses on the culvert foundation and walls.

Three design approaches are commonly applied; designing the culvert with enough strength and rigidity to tolerate these pressures (recognizing that the maximum differential pressures and movements as a result of frost lensing cannot be accurately quantified); removing the frost susceptible soils within the frost zone; or providing adequate insulation to reduce frost penetration. As the frost penetration is extended below the invert level of the culvert, the frost protection should be in accordance with OPSD 803.010 "Backfill and Cover for Concrete Culverts, Frost Penetration Line below Top of Culvert".

If sub-excavation for frost effects is carried out in the dry (with adequate dewatering controls), the material can be replaced with Granular B Type 1 material meeting SSP 110S13 requirements compacted to 95% of standard proctor maximum dry density in accordance with OPSS 501. If the excavation is in the wet (water is maintained at or above adjacent groundwater table) then the material should be rockfill or clear stone surrounded by geotextile, without the need for compaction. Depending on the structural design of the culvert, partial sub-excavation (less than 2.6 m) may also be considered to reduce differential stresses associated with frost; however the exact pressures and movements cannot be accurately quantified.

Acceptable insulation to prevent frost penetration would be 150 mm Dow Styrofoam Highload 40 Insulation or an equivalent material with a compressive strength of approximately 275 kPa or greater. For a region that has a freezing index greater than 3000 Fahrenheit Degree-Days it is recommended that the insulation be placed beneath the structure and extend 2.44 m from the concrete face of the buried structure.

5.1.12 Embankment Foreslopes

Existing culvert foreslopes are approximately 1.5H:1V and 2H:1V on the west and east embankment respectively. The foreslopes should be reinstated with a slope not steeper than 2H:1V if being constructed with granular materials. The foreslopes should be reinstated with a slope not steeper than 1.5H:1V if being constructed with rock fill.

5.1.13 Construction Concerns

The main construction issues that need to be addressed for this site are removal of cover/embankment materials, staged removal of the existing culvert, provisions required for temporary roadway protection, diversion of the channel, excavation below the water table and reinstatement of the embankment fill. These items are important for the successful installation of the new culvert. Particular attention should be paid to maintain the integrity of the existing culvert during the staged method of construction as well as the ability of the chosen roadway protection to accommodate the presence of cobbles within the embankment fill.

A Quality Verification Engineer shall be required to inspect the condition of the foundation and surrounding soils before installation of bedding and other backfills and ensure the width of trench and trench slope walls are suitable, and ensure compliance with materials placed and compaction methods.

6. CLOSURE

Based on the information collected from field investigation and parameters interpreted from laboratory test results, groundwater monitoring data and information provided by the client, culvert replacement options considered were replacement with precast concrete culvert with the use of roadway protection in the dry as well as in the wet. Table 6.1 below summarizes the advantages and disadvantages of construction in the dry versus construction in the wet. Table 6.2 below summarizes the advantages and disadvantages of the use of sheet pile roadway protection versus soldier pile roadway protection.

Table 6.1 Advantages and disadvantages comparison of construction in the dry versus in the wet

Replacement Option	Advantages	Disadvantages
Precast Concrete Culvert installed with Roadway Protection in the Dry	<ul style="list-style-type: none"> • Allows for inspection of subgrade • Allows for careful preparation of stream bed • Prevention of migration of fines between culvert sections during installation • Allows for proper sealing of culvert sections • Ease of erosion control 	<ul style="list-style-type: none"> • Additional construction cost • Potential of piping of materials if granular materials encountered and inadequate dewatering design used • Specialized construction and design required
Precast Concrete Culvert installed with Roadway Protection in the Wet	<ul style="list-style-type: none"> • Ease of construction 	<ul style="list-style-type: none"> • Unconfirmed subgrade • Increase in erosion and sedimentation due to flowing water through construction site

Table 6.2 Advantages and disadvantages comparison of sheet pile versus soldier pile roadway protection

Roadway Protection Option	Advantages	Disadvantages
Sheet Pile	<ul style="list-style-type: none"> • Relatively non permeable • Increased erosion control 	<ul style="list-style-type: none"> • Lightweight material may not accommodate presence of cobbles • Higher installation cost • Specialized construction and design required
Soldier Pile	<ul style="list-style-type: none"> • Heavier gauge materials may be better able to accommodate presence of cobbles • Lower cost 	<ul style="list-style-type: none"> • Permeable • Potential for erosion of retained materials • longer installation time

7. REFERENCES

Canadian Highway Bridge Design Code (2006), CAN/CSA-S6-06, A National Standard of Canada, Canadian standards Association.

Municipal and Provincial Common, Volume 1 - General & Construction Specifications, "*Ontario Provincial Standard for Roads & Public Works*" Spec No. OPSS 422, 501, 510, 511, 517, 518, 539, 805, 902.

Municipal and Provincial Common, Volume 3 - Drawings for Roads, Barriers, Drainage, Sanitary Sewers, Watermains and Structures, "*Ontario Provincial Standard for Roads & Public Works*" Spec No. OPSD 203.040, 803.010, 810.010, 810.020, 3090.100.

Municipal and Provincial Common, Volume 2 - Material Specifications, "*Ontario Provincial Standard for Roads & Public Works*" Spec No. OPSS 1860.

Special Provisions, Ontario Provincial Standards, SP110S13, SP105S10.

8. LIMITATIONS OF REPORT

A description of limitations which are inherent in carrying out site investigation studies is given in Appendix 'A', and this forms an integral part of this report.

For DST CONSULTING ENGINEERS INC.

Prepared by:



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Reviewed by:



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Reviewed by:



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Project Manager

APPENDIX 'A'

LIMITATIONS OF REPORT

LIMITATIONS OF REPORT

GEOTECHNICAL STUDIES

The data, conclusions and recommendations which are presented in this report, and the quality thereof, are based on a scope of work authorized by the Client. Note that no scope of work, no matter how exhaustive, can identify all conditions below ground. Subsurface and groundwater conditions between and beyond the testholes may differ from those encountered at the specific locations tested, and conditions may become apparent during construction which were not detected and could not be anticipated at the time of the site investigation. Conditions can also change with time. It is recommended practice that a Quality Verification Engineer be retained during construction to confirm that the subsurface conditions throughout the site do not deviate materially from those encountered in the testholes. The benchmark and elevations used in this report are primarily to establish relative elevation differences between the testhole locations and should not be used for other purposes, such as grading, excavation, planning, development, etc.

The design recommendations given in this report are applicable only to the project described in the text and then only if constructed substantially in accordance with details stated in this report. Since all details of the design may not be known, we recommend that we be retained during the final stage to verify that the design is consistent with our recommendations, and that assumptions made in our analysis are valid.

Unless otherwise noted, the information contained herein in no way reflects on environmental aspects of either the site or the subsurface conditions.

The comments given in this report on potential construction problems and possible methods are intended only for the guidance of the designer. The number of testholes may not be sufficient to determine all the factors that may affect construction methods and costs, e.g. the thickness of surficial topsoil or fill layers may vary markedly and unpredictably. The contractors bidding on this project or undertaking the construction should, therefore, make their own interpretation of the factual information presented and draw their own conclusion as to how the subsurface conditions may affect their work.

Any results from an analytical laboratory or other subcontractor reported herein have been carried out by others, and DST Consulting Engineers Inc. cannot warranty their accuracy. Similarly, DST cannot warranty the accuracy of information supplied by the client.

APPENDIX 'B'
DESCRIPTIVE TERMS
FOR SOIL CLASSIFICATION

Descriptive Terms for soil classification:

As per the soil classification manual by MTO, the descriptive terms based on percent by mass of the whole sample, are described as per following table

Descriptive Term	Example	Percent by Mass of Sample
And (with two major soil types)	Sand and gravel	40-60
Adjective (silty)	Silty	30-40
With	Silt with fine sand	20-30
Some	Silt, some fine sand	10-20
Trace	Sand, trace of gravel	0-10

EXPLANATION OF TERMS USED IN REPORT

N VALUE: THE STANDARD PENETRATION TEST (SPT) N VALUE IS THE NUMBER OF BLOWS REQUIRED TO CAUSE A STANDARD 51mm O.D. SPLIT BARREL SAMPLER TO PENETRATE 0.3m INTO UNDISTURBED GROUND IN A BOREHOLE WHEN DRIVEN BY A HAMMER WITH A MASS OF 63.5kg, FALLING FREELY A DISTANCE OF 0.76m. FOR PENETRATIONS OF LESS THAN 0.3m N VALUES ARE INDICATED AS THE NUMBER OF BLOWS FOR THE PENETRATION ACHIEVED. AVERAGE N VALUE IS DENOTED THUS \bar{N} .

DYNAMIC CONE PENETRATION TEST: CONTINUOUS PENETRATION OF A CONICAL STEEL POINT (51mm O.D. 60° CONE ANGLE) DRIVEN BY 475 J IMPACT ENERGY ON 'A' SIZE DRILL RODS. THE RESISTANCE TO CONE PENETRATION IS MEASURED AS THE NUMBER OF BLOWS FOR EACH 0.3m ADVANCE OF THE CONICAL POINT INTO THE UNDISTURBED GROUND.

SOILS ARE DESCRIBED BY THEIR COMPOSITION AND CONSISTENCY OR DENSENESS.

CONSISTENCY: COHESIVE SOILS ARE DESCRIBED ON THE BASIS OF THEIR UNDRAINED SHEAR STRENGTH (c_u) AS FOLLOWS:

c_u (kPa)	0 - 12	12 - 25	25 - 50	50 - 100	100 - 200	> 200
	VERY SOFT	SOFT	FIRM	STIFF	VERY STIFF	HARD

DENSENESS: COHESIONLESS SOILS ARE DESCRIBED ON THE BASIS OF DENSENESS AS INDICATED BY SPT N VALUES AS FOLLOWS:

N (BLOWS/0.3m)	0 - 5	5 - 10	10 - 30	30 - 50	> 50
	VERY LOOSE	LOOSE	COMPACT	DENSE	VERY DENSE

ROCKS ARE DESCRIBED BY THEIR COMPOSITION AND STRUCTURAL FEATURES AND / OR STRENGTH.

RECOVERY: SUM OF ALL RECOVERED ROCK CORE PIECES FROM A CORING RUN EXPRESSED AS A PERCENT OF THE TOTAL LENGTH OF THE CORING RUN.

MODIFIED RECOVERY: SUM OF THOSE INTACT CORE PIECES, 100mm+ IN LENGTH EXPRESSED AS A PERCENT OF THE LENGTH OF THE CORING RUN. THE ROCK QUALITY DESIGNATION (RQD), FOR MODIFIED RECOVERY, IS:

RQD (%)	0 - 25	25 - 50	50 - 75	75 - 90	90 - 100
	VERY POOR	POOR	FAIR	GOOD	EXCELLENT

JOINTING AND BEDDING:

SPACING	50mm	50 - 300mm	0.3m - 1m	1m - 3m	> 3m
JOINTING	VERY CLOSE	CLOSE	MOD. CLOSE	WIDE	VERY WIDE
BEDDING	VERY THIN	THIN	MEDIUM	THICK	VERY THICK

ABBREVIATIONS AND SYMBOLS

FIELD SAMPLING

S S	SPLIT SPOON	T P	THINWALL PISTON
W S	WASH SAMPLE	O S	OSTERBERG SAMPLE
S T	SLOTTED TUBE SAMPLE	R C	ROCK CORE
B S	BLOCK SAMPLE	P H	T W ADVANCED HYDRAULICALLY
C S	CHUNK SAMPLE	P M	T W ADVANCED MANUALLY
T W	THINWALL OPEN	F S	FOIL SAMPLE

MECHANICAL PROPERTIES OF SOIL

m_v	kPa^{-1}	COEFFICIENT OF VOLUME CHANGE
C_c	1	COMPRESSION INDEX
C_s	1	SWELLING INDEX
C_α	1	RATE OF SECONDARY CONSOLIDATION
c_v	m^2/s	COEFFICIENT OF CONSOLIDATION
H	m	DRAINAGE PATH
T_v	1	TIME FACTOR
U	%	DEGREE OF CONSOLIDATION
σ'_{v0}	kPa	EFFECTIVE OVERBURDEN PRESSURE
σ'_p	kPa	PRECONSOLIDATION PRESSURE
τ_f	kPa	SHEAR STRENGTH
c'	kPa	EFFECTIVE COHESION INTERCEPT
ϕ'	-°	EFFECTIVE ANGLE OF INTERNAL FRICTION
c_u	kPa	APPARENT COHESION INTERCEPT
ϕ_u	-°	APPARENT ANGLE OF INTERNAL FRICTION
τ_R	kPa	RESIDUAL SHEAR STRENGTH
τ_r	kPa	REMOULDED SHEAR STRENGTH
S_t	1	SENSITIVITY = $\frac{c_u}{\tau_r}$

STRESS AND STRAIN

u_w	kPa	PORE WATER PRESSURE
r_u	1	PORE PRESSURE RATIO
σ	kPa	TOTAL NORMAL STRESS
σ'	kPa	EFFECTIVE NORMAL STRESS
τ	kPa	SHEAR STRESS
$\sigma_1, \sigma_2, \sigma_3$	kPa	PRINCIPAL STRESSES
ϵ	%	LINEAR STRAIN
$\epsilon_1, \epsilon_2, \epsilon_3$	%	PRINCIPAL STRAINS
E	kPa	MODULUS OF LINEAR DEFORMATION
G	kPa	MODULUS OF SHEAR DEFORMATION
μ	1	COEFFICIENT OF FRICTION

PHYSICAL PROPERTIES OF SOIL

ρ_s	kg/m^3	DENSITY OF SOLID PARTICLES	e	1, %	VOID RATIO	e_{\min}	1, %	VOID RATIO IN DENSEST STATE
γ_s	KN/m^3	UNIT WEIGHT OF SOLID PARTICLES	n	1, %	POROSITY	I_D	1	DENSITY INDEX = $\frac{e_{\max} - e}{e_{\max} - e_{\min}}$
ρ_w	kg/m^3	DENSITY OF WATER	w	1, %	WATER CONTENT	D	mm	GRAIN DIAMETER
γ_w	KN/m^3	UNIT WEIGHT OF WATER	S_r	%	DEGREE OF SATURATION	D_n	mm	n PERCENT - DIAMETER
ρ	kg/m^3	DENSITY OF SOIL	w_L	%	LIQUID LIMIT	C_u	1	UNIFORMITY COEFFICIENT
γ	KN/m^3	UNIT WEIGHT OF SOIL	w_p	%	PLASTIC LIMIT	h	m	HYDRAULIC HEAD OR POTENTIAL
ρ_d	kg/m^3	DENSITY OF DRY SOIL	w_s	%	SHRINKAGE LIMIT	q	m^3/s	RATE OF DISCHARGE
γ_d	KN/m^3	UNIT WEIGHT OF DRY SOIL	I_p	%	PLASTICITY INDEX = $w_L - w_p$	v	m/s	DISCHARGE VELOCITY
ρ_{sat}	kg/m^3	DENSITY OF SATURATED SOIL	I_L	1	LIQUIDITY INDEX = $\frac{w - w_p}{I_p}$	i	1	HYDRAULIC GRADIENT
γ_{sat}	KN/m^3	UNIT WEIGHT OF SATURATED SOIL	I_C	1	CONSISTENCY INDEX = $\frac{w_L - w}{I_p}$	k	m/s	HYDRAULIC CONDUCTIVITY
ρ'	kg/m^3	DENSITY OF SUBMERGED SOIL	e_{\max}	1, %	VOID RATIO IN LOOSEST STATE	j	KN/m^2	SEEPAGE FORCE
γ'	KN/m^3	UNIT WEIGHT OF SUBMERGED SOIL						

APPENDIX 'C'

GENIVAR STAGING DRAWINGS

A P P E N D I X ‘ D ‘
N O N - S T A N D A R D
S P E C I A L P R O V I S I O N S

NOTICE TO CONTRACTOR

Special Provision

FOUNDATION CONDITIONS

The Contractor is advised of the following foundation conditions:

Peterson Creek Culvert (Site # 39E-257)

Cobbles and rock fill were identified within the sand fill layer within the advanced borehole locations.

The foundation soils, sensitive soil in particular, will be very susceptible to disturbance and weakening as a result of traffic, standing water and frost. Any foundation soils that could be disturbed should be protected. The bottom of the excavation on which the culvert or granular pad is to rest shall not be disturbed. The bedding placement shall commence immediately after the final removal of material to the foundation level has been completed.

The contractor shall be notified of the high water table and surface water elevation as noted in the Foundation Investigation Report for Peterson Creek.

Heighington Creek Culvert (Site # 39E-198)

Cobbles and rock fill were identified within the sand fill layer within the advanced borehole locations.

The foundation soils, sensitive soil in particular, will be very susceptible to disturbance and weakening as a result of traffic, standing water and frost. Any foundation soils that could be disturbed should be protected. The bottom of the excavation on which the culvert or granular pad is to rest shall not be disturbed. The bedding placement shall commence immediately after the final removal of material to the foundation level has been completed.

The contractor shall be notified of the high water table and surface water elevation as noted in the Foundation Investigation Report for Heighington Creek.

DEWATERING STRUCTURE EXCAVATION - Item No.

Special Provision

902.01 SCOPE

Section OPSS 902.01 of OPSS 902 is amended by the addition of the following:

As part of the work under this item, the Contractor shall:

- Carry out any additional field investigation the Contractor deems necessary in order to engineer the dewatering systems;
- Design and install dewatering systems to construct the work in the dry;
- Provide temporary bypass for watercourse;
- Carry out works necessary for the dewatering system that may include fish salvage/relocation, sheet piling, tremie concrete seal, sand bagging, etc.;

The Contractor shall provide a continuous dewatering operation to keep the excavation stable and free of water. The excavation must be monitored throughout the duration of excavation until the completion of backfilling. The dewatering system must be maintained and the surrounding area monitored for impacts to items such as, but not limited to, settlement and groundwater usage.

The contractor shall also maintain flow in watercourse through the use of a temporary water bypass system which shall be designed to accommodate the design flow rate. The design flows are provided in the contract drawings.

Fish are resident year round in this water body. Wherever a pump is used for dewatering in an area where there possibly may be fish the pump inlet must be suitably screened (with 30 mm clear stone or equivalent) to prevent fish entrainment.

This item includes all installation, modification, and removal of the dewatering system and temporary water passage system as outlined in Operational Constraints: Waterbody/Fisheries in Waterbodies and on Waterbody Banks. All additional excavation and backfill, roadway protection or other temporary works required to provide the temporary bypass shall be included.

The Contractor must satisfy himself with the local conditions and anticipated water flows, levels and flow velocity to be met with during construction. He shall make his own estimate of the facilities required and difficulties to be encountered including the nature of subsurface materials and conditions.

902.03 DEFINITIONS

Section OPSS 902.03 of OPSS 902 is amended by the addition of the following definitions and the deletion of the current definitions of these items, as applicable:

Stamped:	Refers to drawings or details that have been reviewed and stamped “Conforms With Contract Documents”. The stamp shall include the date and signature of the Quality Verification Engineer (QVE).
Quality Verification Engineer (QVE):	An Engineer licensed to practice in the Province of Ontario who has a minimum of five (5) years of experience in the field of design and/or construction of dewatering systems. The Contractor shall retain the QVE to ensure conformance with the contract document.
Dewatering System Design Engineer:	An Engineer licensed to practice in the Province of Ontario who has a minimum of five (5) years of experience in the field of design and/or construction of bridges. In addition, the Dewatering System Design Engineer shall have had responsible experience in the design of at least 5 other dewatering systems. The Contractor shall retain the Dewatering System Design Engineer to ensure conformance with the contract documents and issue certificate(s) of conformance for the design.
Certificate of Conformance	The certificate of conformance shall mean a document issued by the dewatering system design engineer confirming that the specified components of the work are in general conformance with the requirements of the contract documents. Certificate shall be signed and sealed by the Dewatering System Design Engineer.

902.04 SUBMISSION AND DESIGN REQUIREMENTS

Section OPSS 902.04 of OPSS 902 is amended by the addition of the following:

Design of components of the dewatering systems shall be in accordance with CAN/CSA-S6-00 and standard referenced therein.

Submission of Shop Drawings

All shop drawings submissions shall bear the seal and signature of the Dewatering System Design Engineer.

The Contractor shall submit to the Quality Verification Engineer shop drawings for review and stamping.

At least two weeks prior to the commencement of dewatering system construction, the Contractor shall submit to the Contract Administrator, for information purposes only, four (4)

sets of stamped drawings/calculations of the dewatering system.

The Contractor shall, at least three (3) weeks prior to the commencement of the dewatering system installation, submit to the QVE for review, four sets of drawings and calculations indicating:

- the dewatering system design, including design criteria and loading;
- the location, type and dimensions of each dewatering system to be used;
- a schematic showing the configuration of all dewatering systems;
- the material and dimensions of dewatering system components to ensure stability of the design excavation and the dewatering system, and the construction sequence and schedule of each component for which the dewatering system is designed.

The QVE shall review all calculations, construction details, shop drawings and procedures.

All submissions shall bear the seal and signature of the Dewatering System Design Engineer and QVE.

Certificates of Conformance

The Dewatering System Design Engineer shall inspect the installation of each component prior to the executing of the next stage in that dewatering system. After the installation/construction of each component, the Contractor shall submit a Certificate of Conformance to the Contract Administrator, sealed and signed by the Dewatering System Design Engineer. The Certificates of Conformance shall state that the dewatering system is in place, and has been installed in conformance with the stamped shop drawings and the Contract Drawings.

The Contractor will note that several Certificates of Conformance may be required, each to coincide with each dewatering system installation.

902.07 CONSTRUCTION

Section OPSS 902.07 of OPSS 902 is amended by the addition of the following:

All concrete work must be carried out in the dry.

Minimum dimensions for the inside face of the dewatering system shall be sufficient for installation of the new culvert.

902.07.04 Dewatering Structure Excavation

Section OPSS 902.07.04 of OPSS 902 is amended by the addition of the following:

If the Contractor's design requires any portion of the bypass culvert to be left in place following backfill operations, the bypass culvert shall be installed at a distance from the new culvert sufficient to ensure the new culvert can be properly backfilled. The portion of the bypass culvert left in place shall be filled with lean concrete following decommissioning.

902.07.08 Certificate of Conformance

Section OPSS 902.07.08 of OPSS 902 is deleted.

902.10 BASIS OF PAYMENT

Section OPSS 902.10 of OPSS 902 is amended by the addition of the following:

Payment at the contract price for the dewatering systems shall be full compensation for all labour, equipment and materials to carry out the work.

CLAY SEAL - Item No.

Special Provision

OPSS 902 shall apply as amended herein:

902.01 SCOPE

Section 902.01 shall be amended by the addition of the following:

Peterson Creek Culvert (Site #39E-257)

The Contractor shall utilize an Ethylene Propylene Diene Monomer (EPDM) membrane barrier at the inlet of the culvert.

The EPDM membrane shall conform to ASTM D412 and the following:

Thickness: 60 mils
Minimum Tensile Strength: 1300 psi
Minimum Ultimate Elongation: 300%
Minimum Tear Resistance: 150 lbs/in

The EPDM membrane shall be installed in accordance with the manufacturer's instructions. The membrane shall be securely connected to the fascia of the bottom slab or apron wall and the concrete toe walls or retaining walls as applicable. This connection shall be impermeable. The membrane shall be laid flat over the backfill material and extend 2.0m beyond the extents of the backfill onto native material. The edges of the membrane shall be keyed into the native material 500mm vertically and horizontally and a protective layer of sand backfill, 300mm thick, shall be placed over top of the membrane prior to the placement of scour protection.

Heighington Creek Culvert (Site #39E-198)

Under this Tender Item, the Contractor shall supply and install the clay seal at the inlet of the culvert.

Alternatively the Contractor may substitute an Ethylene Propylene Diene Monomer (EPDM) membrane barrier in its place.

The EPDM membrane shall conform to ASTM D412 and the following:

Thickness: 60 mils
Minimum Tensile Strength: 1300 psi
Minimum Ultimate Elongation: 300%
Minimum Tear Resistance: 150 lbs/in

The EPDM membrane shall be installed in accordance with the manufacturer's instructions. The membrane shall be securely connected to the fascia of the bottom slab or apron wall and the concrete toe walls or retaining walls as applicable. This connection shall be impermeable. The membrane shall be laid flat over the backfill material and extend 2.0m beyond the extents of the backfill onto native material. The edges of the membrane shall be keyed into the native material 500mm vertically and

horizontally and a protective layer of sand backfill, 300mm thick, shall be placed over top of the membrane prior to the placement of scour protection.

3000 MM X 2400 MM PRECAST CONCRETE BOX CULVERT - Item No.

Special Provision

OPSS 422 shall apply except as amended:

422.01 SCOPE

Under this Tender Item, the Contractor shall pick-up at the designated pick-up location, deliver and install the precast concrete box culvert units as shown on the Contract Documents, including the supply and placement of the rigid insulation and the placement of the geotextile at the culvert joints. These tender items also include the fabrication, deliver and installation of the precast concrete cut-off wall as shown on the Contract Drawings.

Peterson Creek Culvert (Site #39E-257) is a 3000 mm x 2400 mm precast concrete box culvert that can be picked up at the following location:

MTO Yard at
Northwest corner of Jct of Hwy 11 and 636, approximately 10 km West of Cochrane.

The yard is owned by MTO adjacent to the highway and does not require any advance notification.

422.03 DEFINITIONS

Section 422.03 shall be amended by the addition of the following paragraph:

Quality Verification Engineer: An engineer who has a minimum of five (5) years experience in the construction and inspection of culverts and associated appurtenances. The Quality Verification Engineer shall be retained by the Contractor to ensure conformance with the contract documents and issue of certificate(s) of conformance.

422.04 SUBMISSION AND DESIGN REQUIREMENTS

Subsection 422.04.01 and 422.04.02 shall be added as follows:

422.04.01 Submission of Shop Drawings

The design and shop drawings shall bear the seal and signature of a Professional Engineer who is licensed by the Association of Professional Engineers of Ontario.

422.04.02 Submission of Erection Procedures

The erection drawings shall bear the seal and signature of a Professional Engineer who is licensed by the Association of Professional Engineers of Ontario.

The Quality Verification Engineer shall affix his seal and signature on the erection procedures verifying that the procedures are consistent with the Contract Documents and sound engineering practices.

The erection procedures shall include at least the following:

1. Lifting points locations
2. Details of all temporary supports

422.05 MATERIALS

422.05.11 Geotextile

Subsection 422.05.11 shall be amended by the addition of the following:

Geotextile to be non-woven, Class II, with a thickness greater than 1mm and a FOS of 50 to 100 microns and shall be according to OPSS 1860.

Section 422.05 shall be amended by the addition of the following subsection:

422.05.16 Rigid Insulation

Rigid Insulation shall be Dow Styrofoam Highload 40 Insulation (minimum compressive strength of 275 kPa) or an equivalent material as per OPSS 1605.

422.07 CONSTRUCTION

OPSS 422.07 shall be amended by the addition of the following:

422.07.01 CCIL Certification

Subsection 422.07.01 shall be amended by the addition of the following paragraphs:

The precast concrete culvert units shall be fabricated by a manufacturer certified in conformance to the "Prequalification Requirements for Manufacturers of Precast Concrete Drainage Products - May 1998", by the MTO/MEA/OCPA/OPS Prequalification Advisory Committee.

Units are to be designed in accordance with the CHBDC 2006 for highway loads with a minimum of 600 mm, to a maximum of 4000 mm of granular fill and roadbase.

422.07.09 Installing Box Units

422.07.09.01 Box Units

Subsection 422.07.09.01 shall be amended by the deletion of the first sentence of the sixth paragraph and the addition of the following:

Installation of the box units shall commence at the downstream end and proceed in the upstream direction with the bell ends of the box units facing upgrade

Subsection 422.07.09.01 shall be amended by the addition of the following paragraphs:

The Contractor shall keep a copy of the signed and sealed erection procedures on the site during erection of the members.

The Contractor shall notify the Contract Administrator in writing of the starting date at least 1 week prior to the commencement of field operations and erection work shall not be carried out until the Contract Administrator is on the site.

Precast concrete cut-off walls units shall be installed to the alignment and grade shown on the Contract Documents. The installation tolerance is ± 5 mm horizontally and vertically.

Precast concrete box culvert units shall be installed to the alignment and grade shown on the Contract Documents. The installation tolerance is ± 5 mm horizontally and vertically.

422.08 QUALITY ASSURANCE

Section 422.08 shall be amended by the addition of the following subsection:

422.08.01 Certificate of Conformance

The Contractor shall submit to the Contract Administrator a certificate of conformance signed and sealed by the Quality Verification Engineer upon completion of each of the following operations and prior to the commencement of each subsequent operation:

- Precast Cut-off Fabrication
- Precast Culvert Installation

The certificate shall state that the work has been executed according to the specification and /or stamped working drawings.

422.10 BASIS OF PAYMENT

Section 422.10 shall be deleted and replaced with the following paragraph:

Payment at the Contract price for the above tender item shall be full compensation for all labour, equipment and material to do the work and includes but not limited to surveying, plastic shims, grouting, geotextile, and rigid insulation.

D R A W I N G S



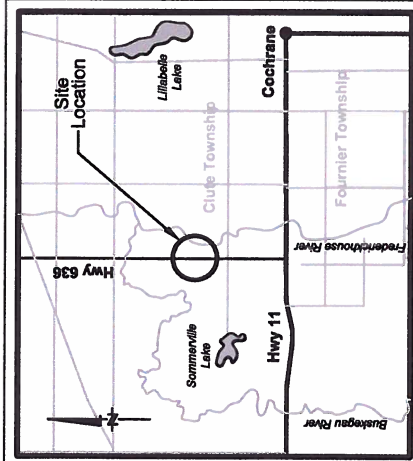
METRIC
DIMENSIONS ARE IN METRES
UNLESS
OTHERWISE SHOWN
DIMENSIONS IN KILOMETRES + METERS



CONT No 2012-5121
GWP No 5481-09-00
WP No 5481-09-01
Site No 39E-257
Geocres No 42H-44

CULVERT REPLACEMENT
3.7 Km North of Hwy 11
Highway 636 – Clute Twp.
Borehole Location Plan

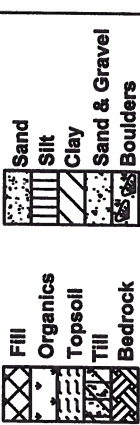
SHEET



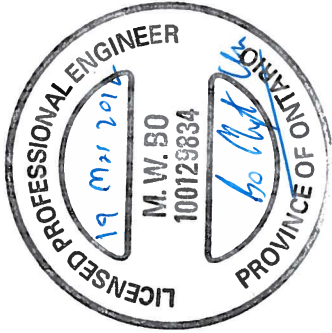
KEY PLAN
SCALE IN KILOMETRES
0 12

LEGEND

- Borehole/Hand Auger
- Borehole with DCPT
- Dynamic Cone Penetration Test (DCPT)
- Rock Probe
- 'N' Blows/0.3m (Std. Pen Test, 475 J/Blow)
- Water level at time of investigation.
- Benchmark



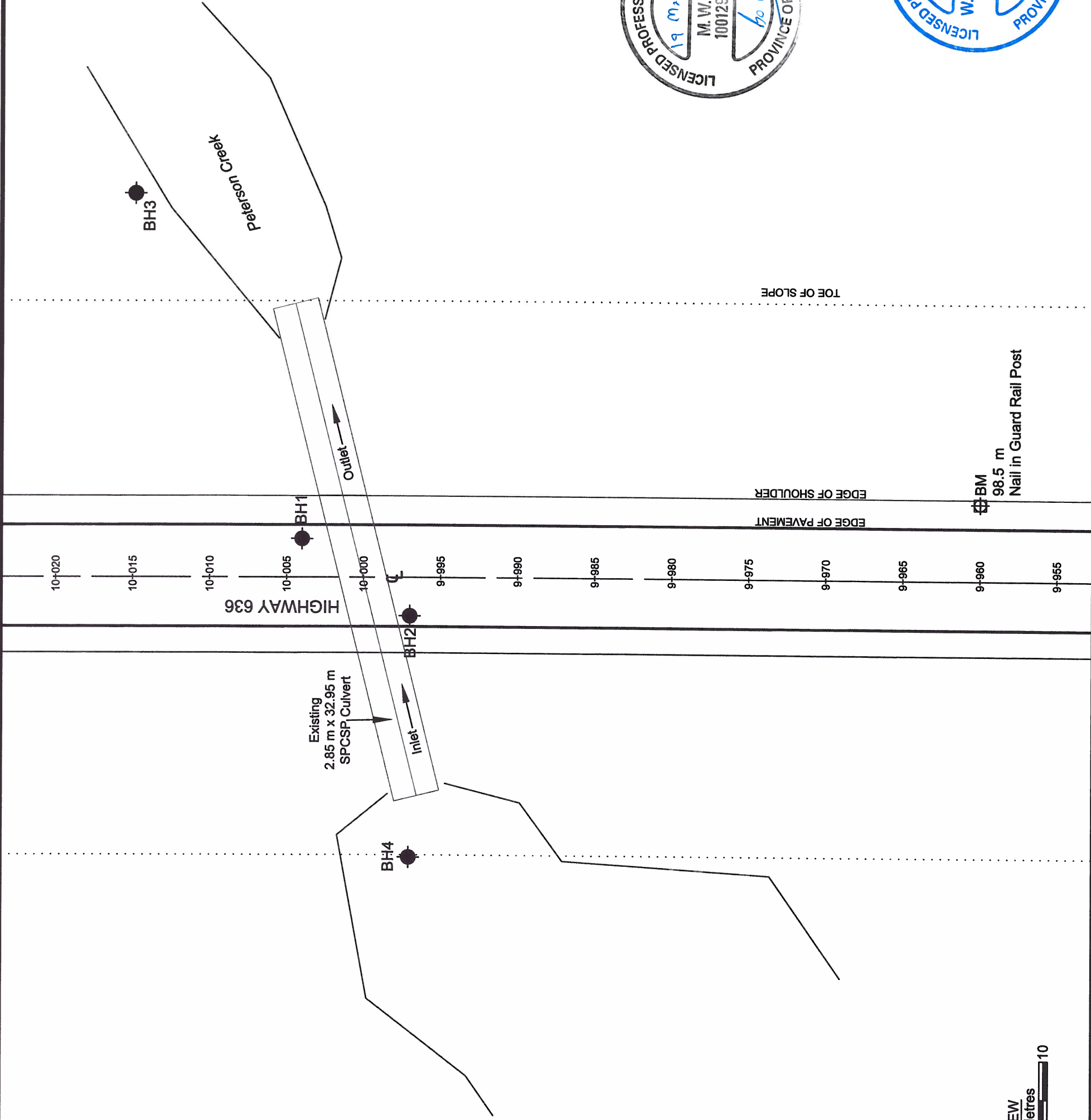
No.	Elevation	Northing	Easting	Station	Offset
BH1	97.22	5437894	488437	10+004	2.5 m RT
BH2	97.34	5437973	488431	9+997	2.5 m LT
BH3	94.05	5437894	488454	10+015	23.0 m RT
BH4	92.15	5437975	488430	9+997	14.0 m LT



NOTE:
The boundaries between soil strata have been established only at borehole locations. Between boreholes the boundaries are assumed by interpolation and may not represent actual conditions.

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605 Hewitson Street
Thunder Bay, ON P7B 5V5
Ph: (807) 623-2929
Fx: (807) 623-1792
Email: thunderbay@dstgroup.com

DRAWING 1



PLAN VIEW
Scale in Metres
0 10

CONT

No 2012-5121

GWP

No 5481-09-00

WP

No 5481-09-01

Site

No 39E-257

Geocres

No 42H-44

CULVERT REPLACEMENT

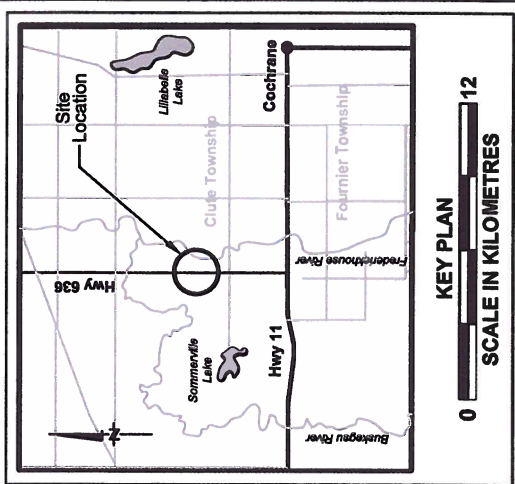
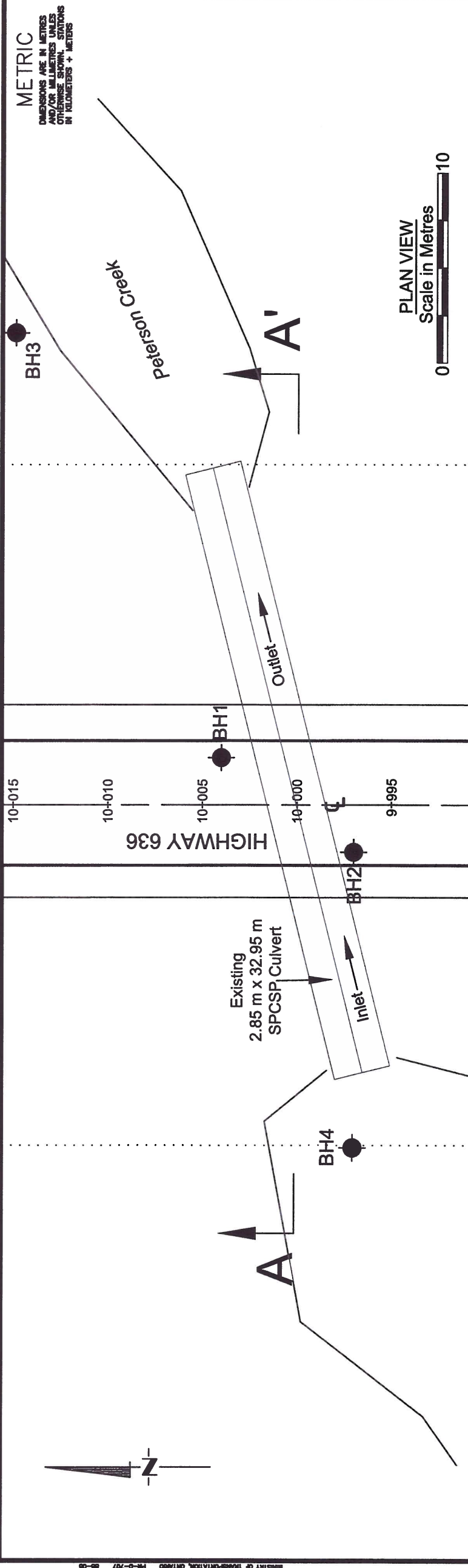
3.7 Km North of Hwy 11

Highway 636 - Clute Twp.

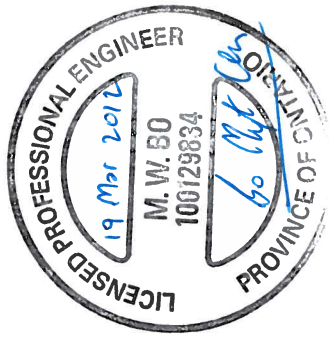
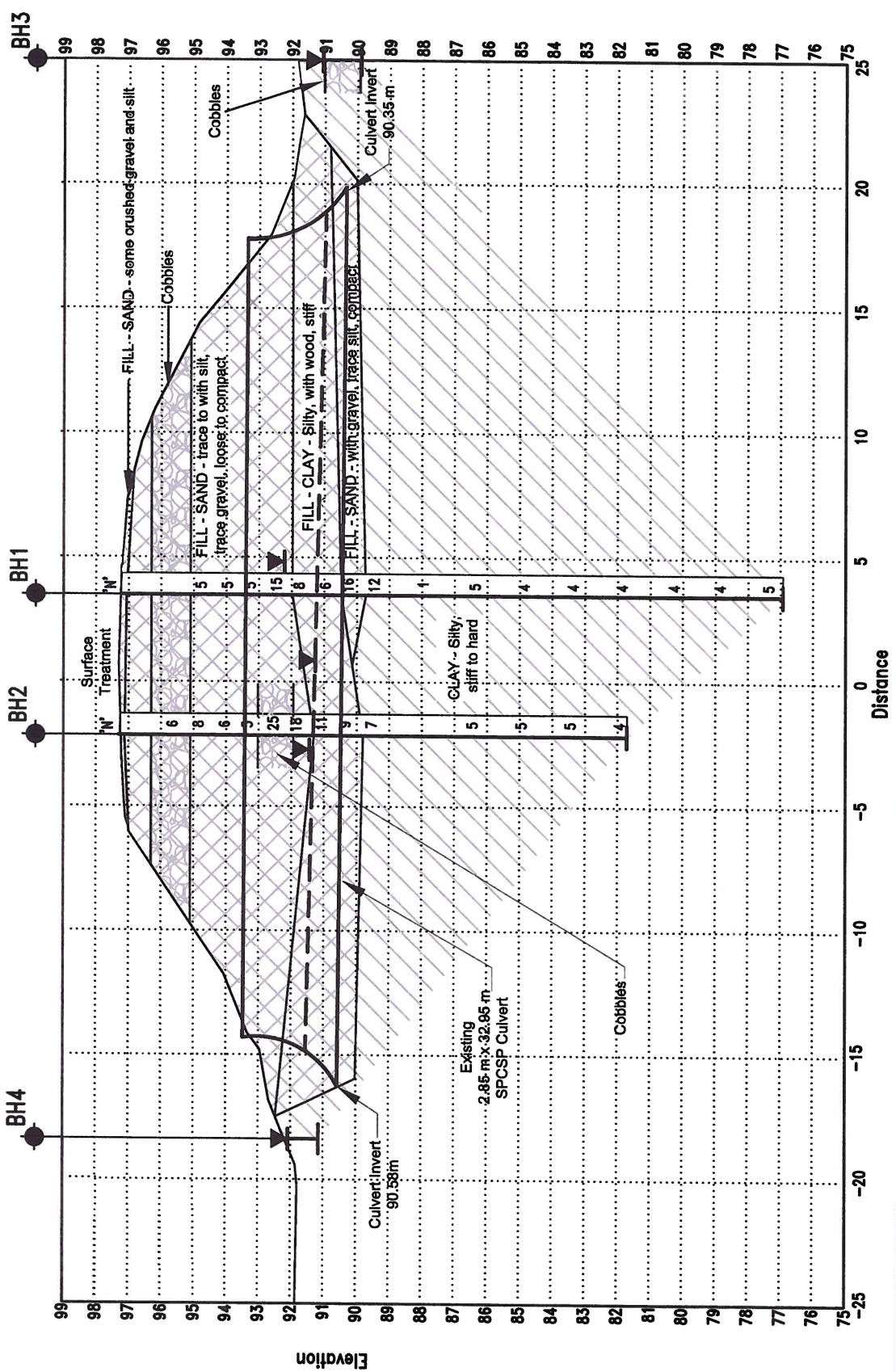
Borehole Location Plan

SHEET

10



Profile Along Section A-A'



NOTE:

The boundaries between soil strata have been established only at borehole locations. Between boreholes the boundaries are assumed by interpolation and may not represent actual conditions.

DST

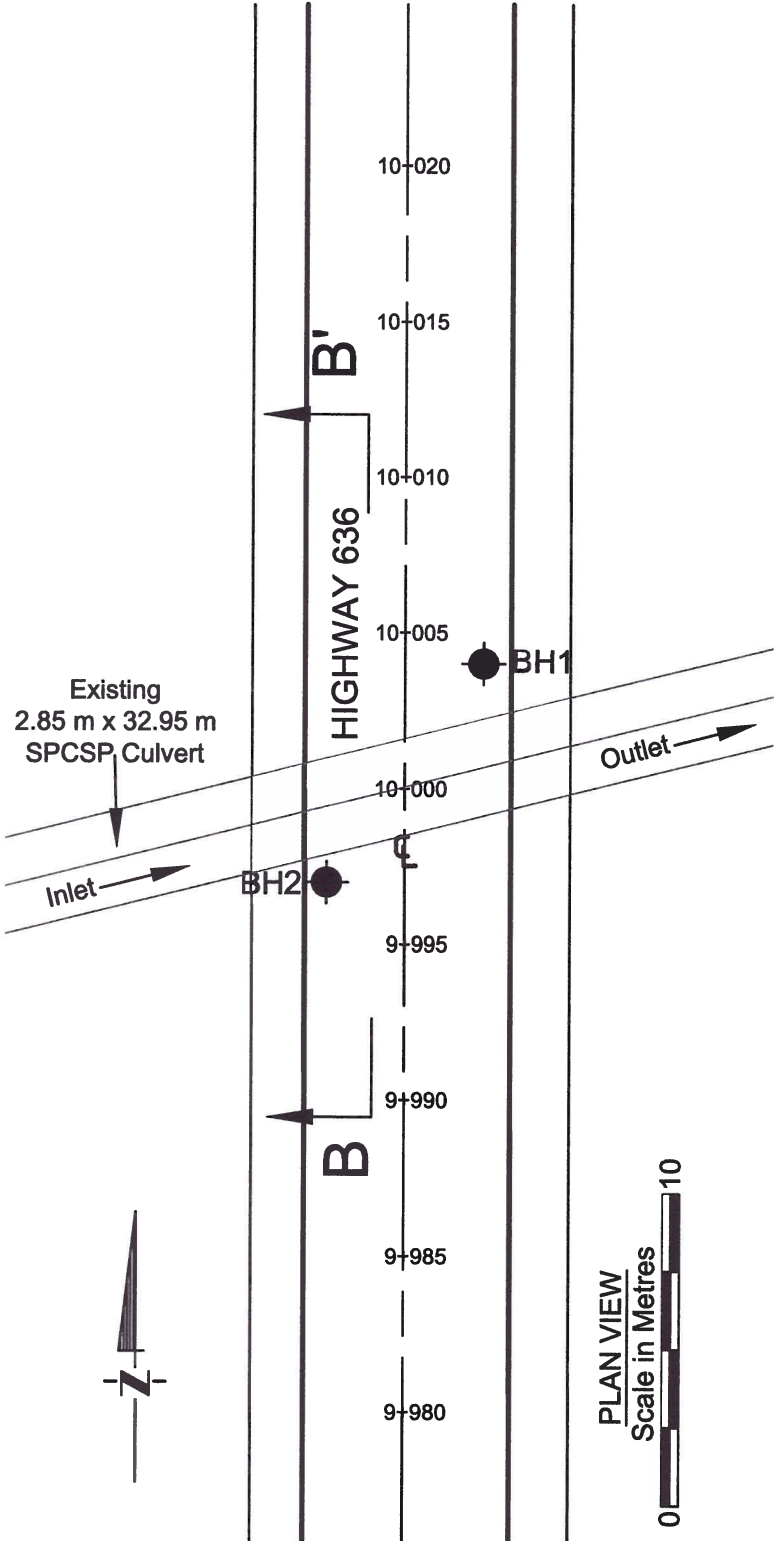
DST Consulting Engineers Inc.
605 Hewitson Street
Thunder Bay, ON P7B 5V5
Ph: (807) 623-2929
Fx: (807) 623-1792
Email: thunderbay@dstgroup.com

METRIC
DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES UNLESS
OTHERWISE SHOWN. STATIONS
IN KILOMETRES + METERS

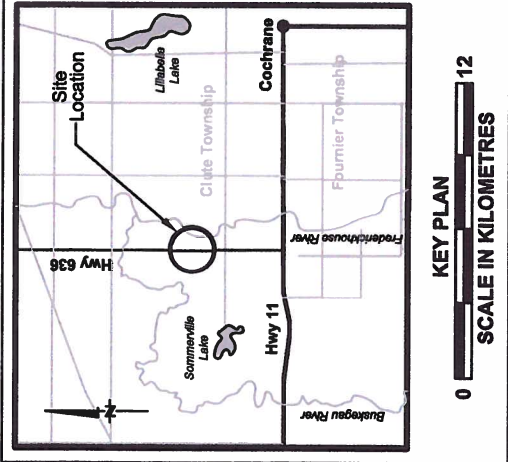
CONT No 2012-5121
GWP No 5481-09-00
WP No 5481-09-01
Site No 39E-257
Geocres No 42H-44

CULVERT REPLACEMENT
3.7 Km North of Hwy 11
Highway 636 - Clute Twp.
Borehole Location Plan

SHEET
11

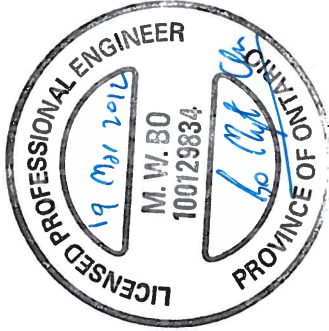


PLAN VIEW
Scale in Metres



KEY PLAN
SCALE IN KILOMETRES

LEGEND					
	Borehole/Hand Auger		Borehole with DCPT		Dynamic Cone Penetration Test (DCPT)
	Rock Probe		Blows/0.3m (Std. Pen Test, 475 J/Blow)		Water level at time of investigation.
	Benchmark		Fill		Organics
	Topsoil		Till		Bedrock
	Sand		Silt		Clay
	Sand & Gravel		Boulders		
No.	Elevation	Northing	Eastng	Station	Offset
BH1	97.22	5437684	488437	10+004	2.5 m RT
BH2	97.34	5437673	488431	9+997	2.5 m LT
BH3	91.05	5437684	488454	10+015	25.0 m RT
BH4	92.15	5437675	488420	9+997	14.0 m LT



NOTE:
The boundaries between soil strata have been established only at borehole
locations. Between boreholes the boundaries are assumed by interpolation
and may not represent actual conditions.

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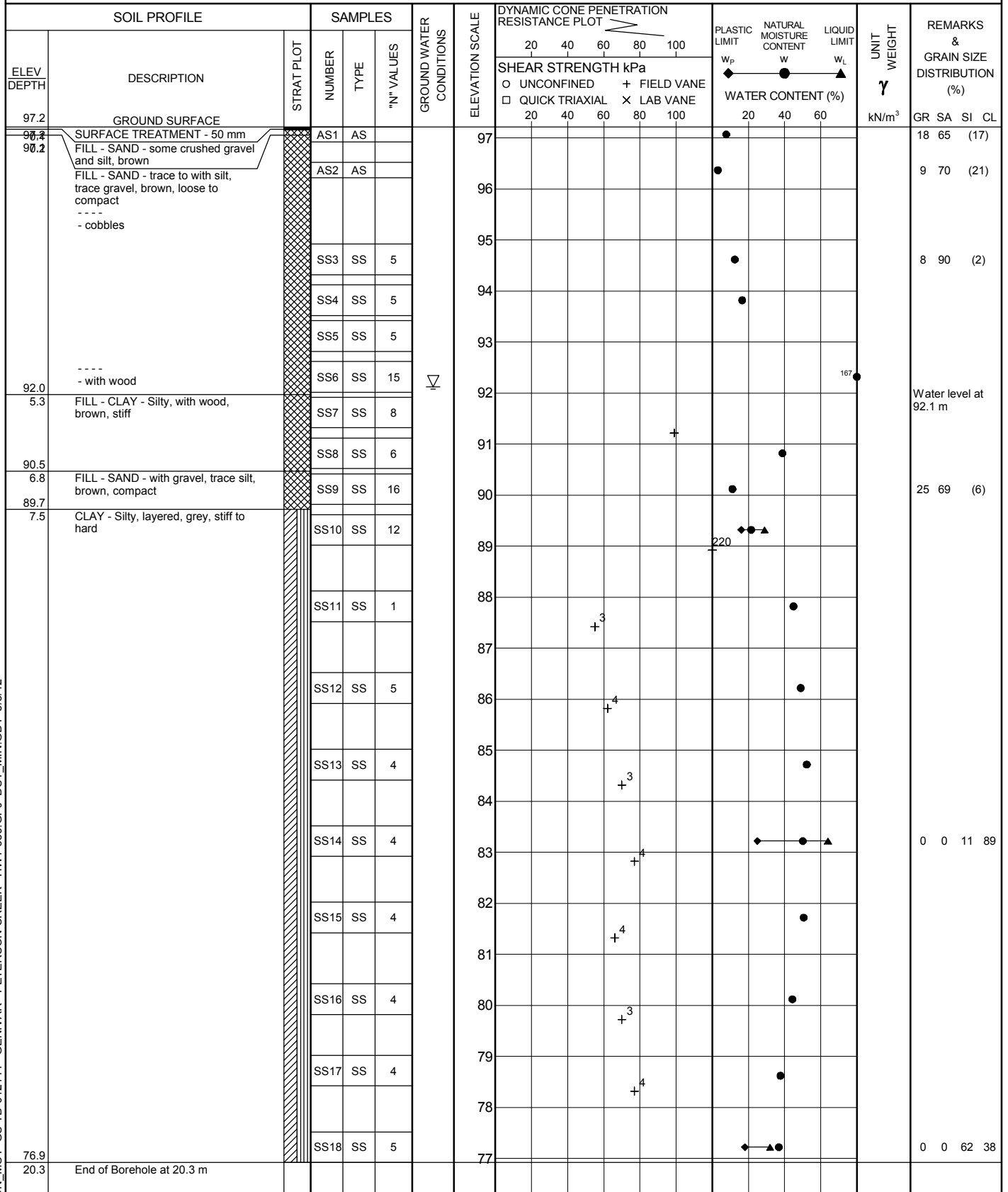
E N C L O S U R E S

RECORD OF BOREHOLE No BH1

1 OF 1

METRIC

W.P. 5481-09-01 LOCATION STA. 10+004, 2.5 m RT (5437684 m N, 488437 m E) ORIGINATED BY PR
DIST HWY 636 BOREHOLE TYPE Hollow Stem Auger COMPILED BY ML
DATUM Local DATE 2011 01 31 CHECKED BY WS/BV



Numbers refer to
Sensitivity

3% STRAIN AT FAILURE

ENCLOSURE 1

METRIC

DN_MOT GS-TB-012144 - GENIVAR - PETERSON CREEK - HWY 636.GPJ DST_MIN.GDT 5/3/12

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 kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							
								○ UNCONFINED	+ FIELD VANE	□ QUICK TRIAXIAL					
97.3	GROUND SURFACE														
98.3	SURFACE TREATMENT - 50 mm														
98.2	FILL - SAND - some crushed gravel and silt, brown		RC1	RC											
	FILL - SAND - trace to some gravel, trace silt, brown, loose to compact														
	- cobbles		SS2	SS	6										1 98 (1)
	- cobbles		SS3	SS	8										
			SS4	SS	6										
			SS5	SS	5										17 77 (6)
	- cobbles		SS6	SS	25										
			SS7	SS	18										
91.3															
6.0	FILL - CLAY - Silty, with wood, brown/grey, stiff		SS8	SS	11										Water level at 91.5 m
			SS9	SS	9										
89.8															
7.5	CLAY - Silty, layered, brown/grey, stiff		SS10	SS	7										
			TW11	TW											
			SS12	SS	5										0 0 63 37
			SS13	SS	5										
			SS14	SS	5										
			SS15	SS	4										
81.7															
15.6	End of Borehole at 15.6 m														0 0 5 95

✕³, ★³: Numbers refer to Sensitivity ○^{3%} STRAIN AT FAILURE

RECORD OF BOREHOLE No BH3

1 OF 1

METRIC

W.P. 5481-09-01 LOCATION STA. 10+015, 25.0 m RT (5437694 m N, 488459 m E) ORIGINATED BY PR
 DIST HWY 636 BOREHOLE TYPE Hand Auger COMPILED BY ML
 DATUM Local DATE 2011 02 05 CHECKED BY WS/BV

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 kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
						20 40 60 80 100 ○ UNCONFINED + FIELD VANE □ QUICK TRIAXIAL × LAB VANE					WATER CONTENT (%)						
						20 40 60 80 100					20 40 60						
91.1	GROUND SURFACE																
90.0	TOPSOIL - 50 mm		AS1	AS													
	CLAY - Silty, trace gravel, brown - cobbles																
89.9			AS2	AS													
1.2	End of Borehole at 1.2 m																

✕³, ★³: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

RECORD OF BOREHOLE No BH4

1 OF 1

METRIC

W.P. 5481-09-01 LOCATION STA. 9+997, 14.0 m LT (5437675 m N, 488420 m E) ORIGINATED BY PR
DIST HWY 636 BOREHOLE TYPE Hand Auger COMPILED BY ML
DATUM Local DATE 2011 02 05 CHECKED BY WS/BV

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	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)				
							20	40	60	80	100	20	40	60			
92.2	GROUND SURFACE																
90.1	TOPSOIL - 60 mm																
91.2	CLAY - Silty, some sand and gravel, brown		AS1	AS												Water level at 92.1 m	
1.0	End of Borehole at 1.0 m																

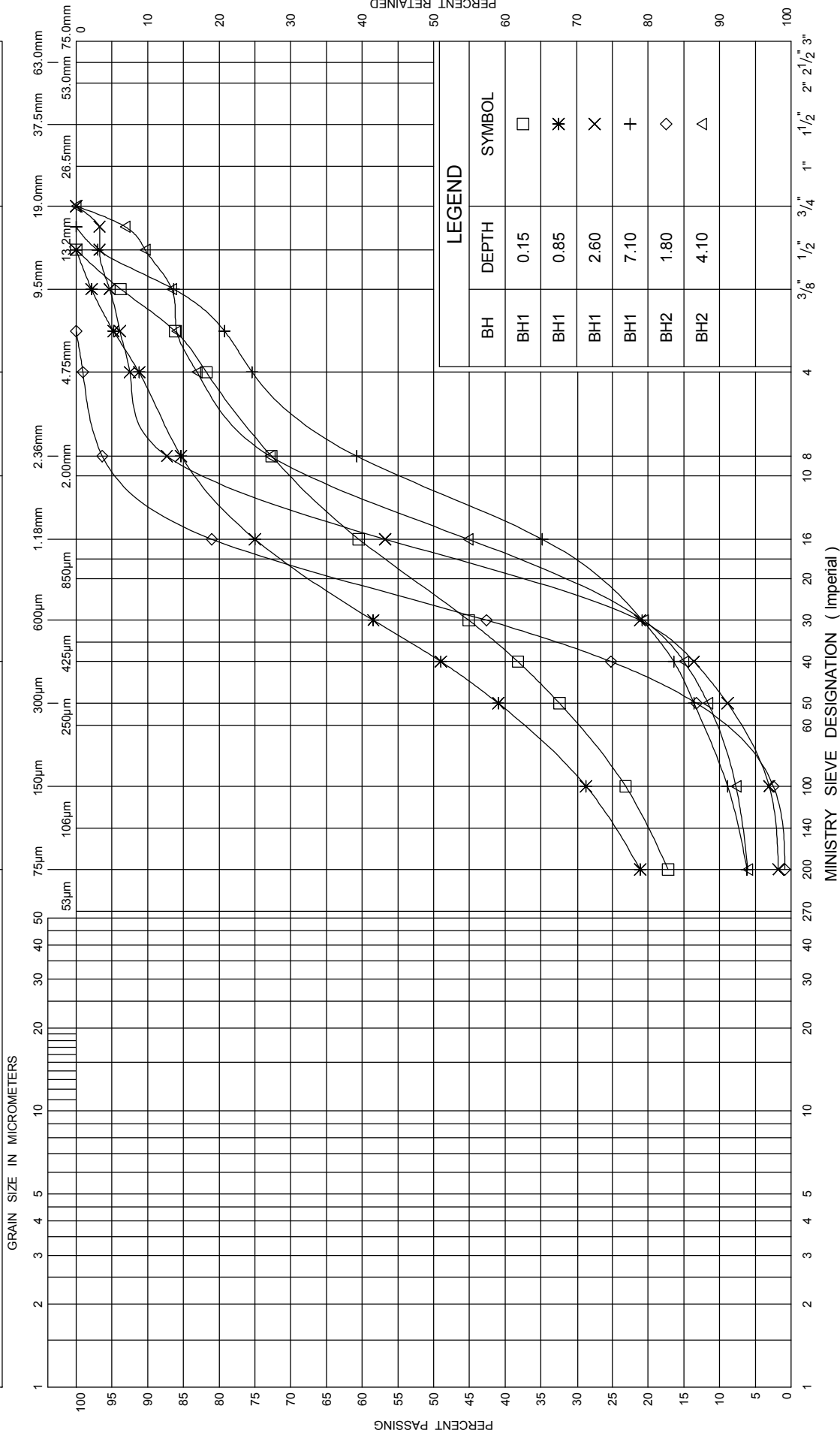
✕³, ★³: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

ENCLOSURE 4

ON_MOT_CS-TB-012144 - GENIVAR - PETERSON CREEK - HWY 636.GPJ DST_MIN.GDT 5/3/12

UNIFIED SOIL CLASSIFICATION SYSTEM

CLAY & SILT		SAND			GRAVEL		
		Fine		Medium	Coarse	Fine	Coarse

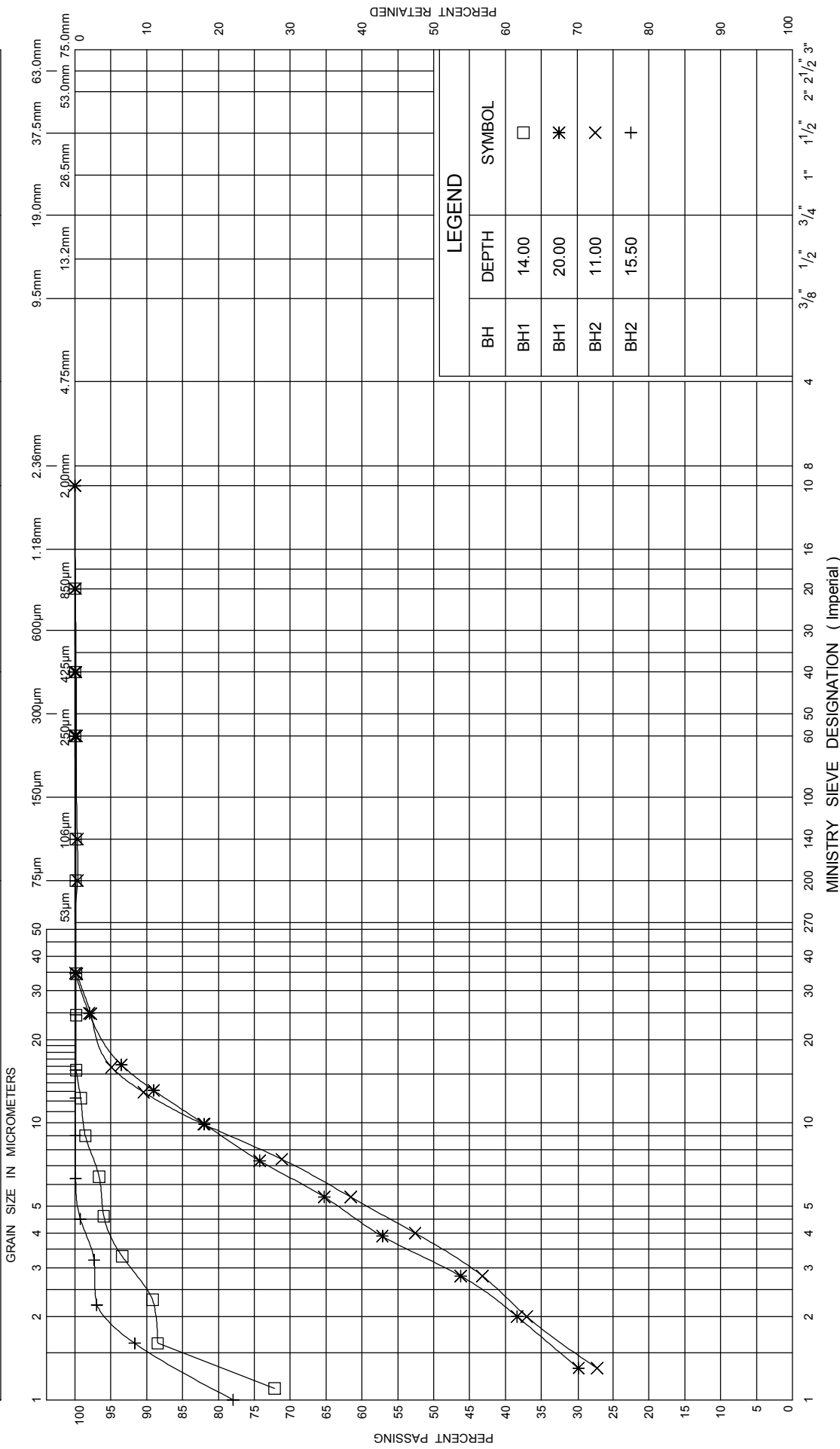


GRAIN SIZE DISTRIBUTION
SAND

ENCLOSURE 5
W P 5481-09-01
HIGHWAY 636

UNIFIED SOIL CLASSIFICATION SYSTEM

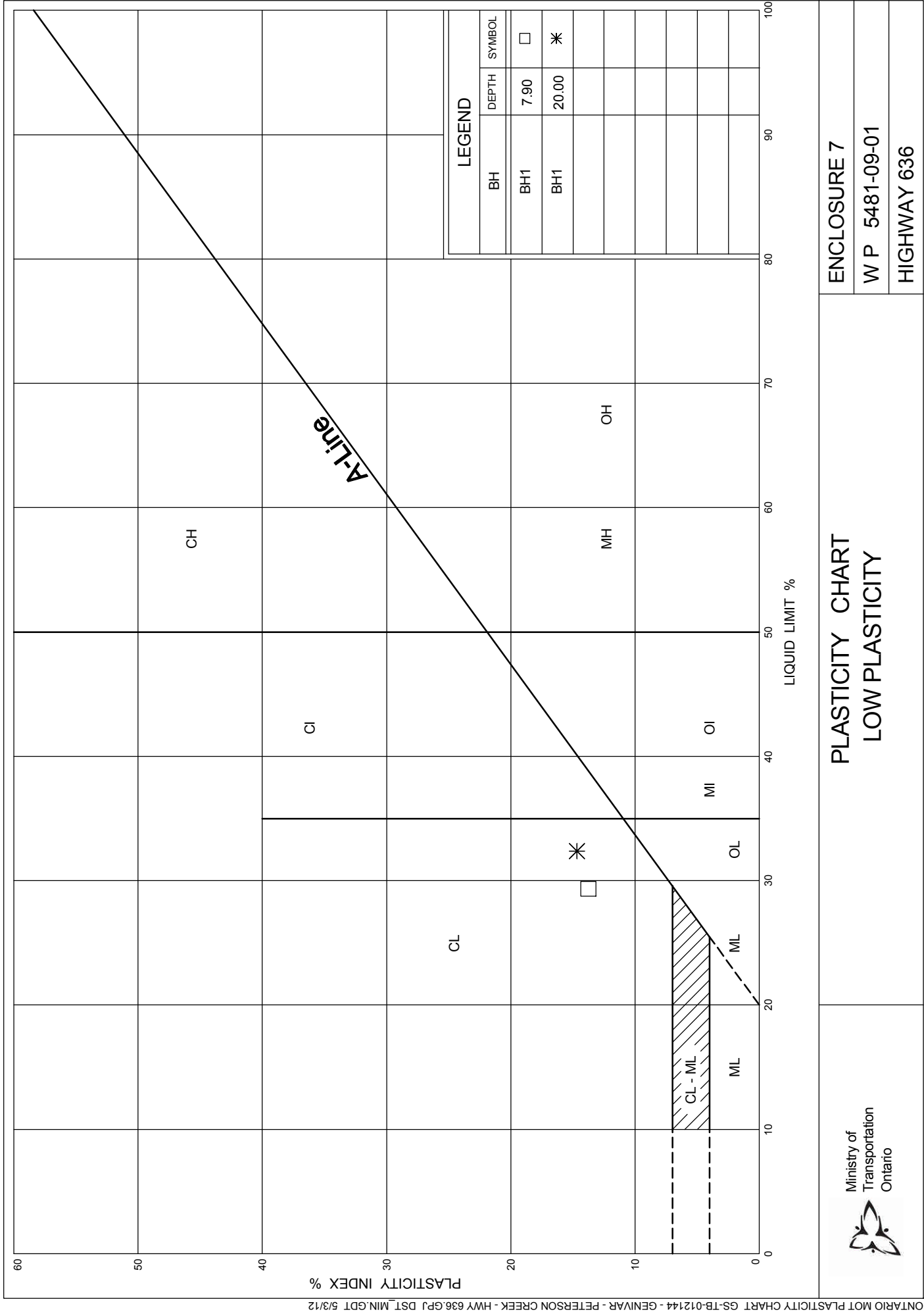
CLAY & SILT		SAND			GRAVEL	
		Fine		Medium	Fine	Coarse



GRAIN SIZE DISTRIBUTION
CLAY

ENCLOSURE 6
W P 5481-09-01
HIGHWAY 636

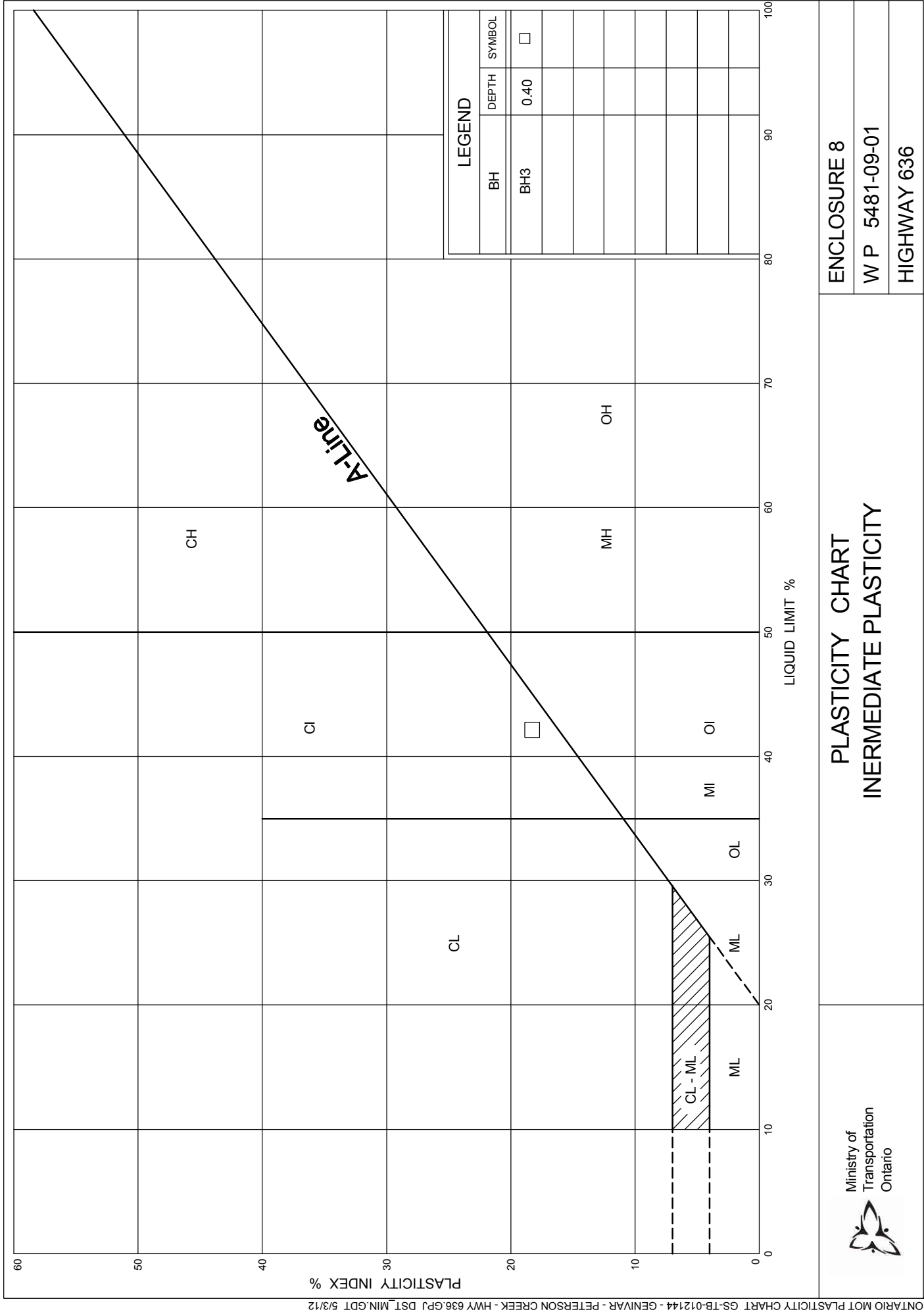




PLASTICITY CHART
LOW PLASTICITY

ENCLOSURE 7
W P 5481-09-01
HIGHWAY 636





PLASTICITY CHART
INTERMEDIATE PLASTICITY

ENCLOSURE 8

W P 5481-09-01

HIGHWAY 636



