

**FOUNDATION INVESTIGATION AND DESIGN REPORT
LITTLE REST CREEK CULVERT REPLACEMENT
HIGHWAY 17, DISTRICT OF THUNDER BAY, ONTARIO
SITE 48W-309/C
G.W.P. 6941-10-00**

Geocres Number: 52B-15

Report to

GENIVAR

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PART 1: FACTUAL INFORMATION

1 INTRODUCTION

This report presents the factual findings obtained from a foundation investigation conducted at the location of a proposed culvert replacement at Little Rest Creek east of Savanne, Ontario. The existing culvert carries Little Rest Creek under Highway 17 in the District of Thunder Bay, Ontario.

The purpose of this investigation was to explore the subsurface conditions at the site and, based on the data obtained, to provide a borehole location plan, records of boreholes, stratigraphic profile, laboratory test results and a written description of the subsurface conditions. A model of the subsurface conditions was developed from the data obtained in the course of the investigation.

Thurber carried out the investigation as a sub-consultant to Genivar, under the Ministry of Transportation Ontario (MTO) Agreement Number 6010-E-0012.

2 SITE DESCRIPTION

The Little Rest Creek culvert is located on Highway 17, approximately 1.4 km east of the Town of Savanne, Ontario. The site is approximately 53.4 km north of the intersection of Highway 17 and Highway 11.

The existing highway is a two-lane paved road and crosses the creek on approach embankments about 3.0 m to 4.0 m high.

Currently twin CSP culverts carry Little Rest Creek under Highway 17. The existing culverts are both 2.4 m in diameter and are approximately 24.4 m long.

Lands surrounding the culvert site consist primarily of forested areas with open swamps.

Photographs in Appendix C show the general nature of the surrounding land.

The site lies within the Superior Province of the Canadian Shield, characterized by low, rounded hills of Pre-Cambrian bedrock mantled by varying thicknesses of overburden. At this site, the overburden primarily consists of glaciolacustrine clay and silty sand.

3 SITE INVESTIGATION AND FIELD TESTING

The site investigation and field testing for this project were carried out on September 17 and 26, 2011 and consisted of drilling and sampling a total of three boreholes (identified as LRC-01 to LRC-03) in the area of the existing culvert. One borehole was drilled near each end of the culvert and one borehole was drilled through the Highway 17 embankment from the north shoulder of the highway. Borehole advancement within the overburden soils extended to depths of 6.2 m to 11.6 m (elevations 448.0 to 451.0).

A dynamic cone penetration test (DCPT) was conducted from the base of Borehole LRC-03 and was terminated at 8.4 m depth (elevation 449.1).

The approximate borehole locations are shown on the attached Borehole Locations and Soil Strata Drawing in Appendix F.

The borehole locations were marked in the field and utility clearances were obtained prior to drilling.

For Borehole LRC-02, located on the highway, drilling was carried out using a truck mounted CME 75 drill rig. Hollow-stem augers were used to advance the borehole. For Boreholes LRC-01 and LRC-03, located near the toes of the highway embankment, drilling was carried out using portable drilling equipment mounted on a tripod. Wash boring techniques were used to advance Boreholes LRC-01 and LRC-03. Overburden samples were obtained at selected intervals using a split spoon sampler in conjunction with Standard Penetration Testing (SPT). In situ vane shear testing was carried out to assess the undrained shear strength of soft to firm cohesive deposits.

The drilling and sampling operations were supervised on a full time basis by a member of Thurber's technical staff. The supervisor logged the boreholes and processed the recovered soil samples for transport to Thurber's laboratory for further examination and testing.

Groundwater conditions were observed in the open boreholes upon completion of the drilling operations. Two standpipe piezometers, consisting of 19 mm diameter PVC pipe with a slotted screen and enclosed in filter sand, were installed in Boreholes LRC-01 and LRC-03 to permit longer term groundwater level monitoring. The boreholes were abandoned in general accordance with O.Reg. 903 upon completion. The completion details of the boreholes and locations of the piezometers and boreholes are shown in Table 3.1.

Table 3.1 – Borehole Abandonment Details

Borehole	Piezometer Tip Depth/ Elevation (m)	Abandonment Details
LRC-01	6.2 / 451.0	Piezometer with 1.5 m slotted screen installed with sand filter to 4.3 m and bentonite holeplug from 4.3 m to surface.
LRC-02	None installed	Backfilled with bentonite holeplug from 11.6 m to 0.3 m, then sand and gravel to surface.
LRC-03	7.0 / 450.5	Piezometer with 1.5 m slotted screen installed with sand filter to 4.9 m and bentonite holeplug from 4.9 m to surface.

An attempt was made to decommission the piezometers in October 2012. However, the water in the piezometers installed in Boreholes LRC-01 and LRC-02 was frozen and it was not possible to decommission them. These piezometers will be decommissioned in the Spring of 2013.

4 LABORATORY TESTING

The recovered soil samples were subjected to Visual Identification (VI) and to natural moisture content determination. Selected samples were also subjected to gradation analysis (sieve and hydrometer) and Atterberg Limits testing, where appropriate. The results of these tests are summarized on the Record of Borehole sheets included in Appendix A and are presented on the figures included in Appendix B.

5 DESCRIPTION OF SUBSURFACE CONDITIONS

Reference is made to the Record of Borehole sheets included in Appendix A. Details of the encountered soil stratigraphy are presented in these sheets and on the “Borehole Locations and Soil Strata” drawing included in Appendix F. An overall description of the stratigraphy is given in the following paragraphs. However, the factual data presented in the Record of Borehole sheets governs any interpretation of the site conditions.

In general terms, the site was found to be underlain by surficial topsoil at the toe of the highway embankment and the borehole drilled through the highway embankment encountered sand fill with gravel. Layers of native silty clay and silty sand were encountered below the topsoil and the embankment fill. Auger refusal on boulders or probable bedrock was encountered below the native silty sand.

More detailed descriptions of the individual strata are presented below.

5.1 Topsoil/Organics

Topsoil and organics were encountered at the surface in Boreholes LRC-01 and LRC-03, which were drilled at the toe of the highway embankment, near the north and south ends of

the existing culvert, respectively. The topsoil was dark brown to brown in colour and contained organics, some clay and occasional roots. The topsoil was 0.6 m thick in Borehole LRC-01 and 1.2 m thick in Borehole LRC-03.

The elevations of the base of the topsoil layer in the boreholes are 456.6 and 456.3.

The topsoil thickness may vary between and beyond the borehole locations.

5.2 Gravelly Sand Fill

Gravelly sand fill was encountered from the surface in Borehole LRC-02, which was drilled through the existing Highway 17 westbound shoulder. The sand fill was brown in colour and contained some gravel to being gravelly with trace silt and clay. The thickness of the sand fill was 4.0 m.

The depth to the base of the fill was 4.0 m (elevation 455.6).

Standard Penetration Tests performed in the sand fill layer gave SPT N-values ranging from 13 to 39 blows for 0.3 m penetration, indicating a compact to dense relative density. The density of the fill decreased with depth.

The moisture content of samples of the sand fill ranged from 3% to 13%.

One sample of the sand fill underwent gradation analysis testing, the results of which are presented below. These results are also summarized on the Record of Borehole sheets in Appendix A and the grain size distribution curve for this sample is plotted on Figure B1 of Appendix B.

Soil Particles	Percentage (%)
Gravel	26
Sand	66
Silt and Clay	8

5.3 Silty Clay

A layer of silty clay was encountered below the topsoil in Boreholes LRC-01 and LRC-03 and below the sand fill in Borehole LRC-02. The silty clay was grey to reddish brown in colour and contained trace sand and gravel and occasional roots. The thickness of the silty clay layer ranged from 3.5 m to 4.3 m.

The depth to the base of the silty clay layer ranged from 4.9 m to 7.5 m (elevations 452.3 to 452.0).

SPT N-values recorded in the silty clay layer ranged from 2 to 7 blows for 0.3 m penetration, indicating a soft to firm consistency. Shear Vane Tests were also performed

where low N-values were recorded. The shear strength of the silty clay ranged from 24 to 44 kPa.

The moisture content of samples of the silty clay ranged from 23% to 99%, typically greater than 40%.

Three samples of the silty clay were selected for grain size analysis testing and two samples were selected for Atterberg Limits testing. The results of these tests are presented on the Record of Borehole sheets included in Appendix A. The grain size distribution curves for these samples are plotted on Figure B2, Appendix B and the results of the Atterberg Limits tests are plotted on Figure B4, Appendix B. The results of the laboratory tests are summarized as follows:

Soil Particles	Percentage (%)
Gravel	0
Sand	0
Silt	27 to 68
Clay	32 to 72

Index Property	Percentage (%)
Liquid Limit	51 to 58
Plastic Limit	20 to 21
Plasticity Index	31 to 37

Results of the Atterberg Limits tests indicate that the silty clay is of high plasticity with a group symbol of CH.

5.4 Silty Sand

Silty sand was encountered below the silty clay in all three boreholes drilled at this site. The silty sand was grey in colour and contained trace to some gravel and trace silt and clay. Occasional cobbles and boulders were encountered within the silty sand in Borehole LRC-02 at a depth of 10.1 m. Bedrock fragments were encountered in Borehole LRC-01 near 6.0 m depth.

The thickness of the silty sand layer penetrated in the three boreholes ranged from 1.3 m to 4.1 m, however all the boreholes were terminated upon refusal below the silty sand layer.

SPT N-values recorded in the silty sand typically ranged from 19 to 74 blows for 0.3 m penetration, indicating a compact to very dense relative density. An SPT N-value of 8 blows for 0.3 m penetration was recorded in Borehole LRC-02 at a depth of 8.0 m,

indicating a loose relative density. One SPT-N value of 50 blows without penetration was recorded in Borehole LRC-02 near elevation 449.0.

The moisture content of samples of the silty sand ranged from 8% to 13%.

Four samples of the silty sand underwent laboratory gradation analysis, the results of which are summarized below. These results are also presented on the Record of Borehole sheets included in Appendix A and the grain size distribution curves for these samples are plotted on Figure B3, Appendix B.

Soil Particles	Percentage (%)
Gravel	8 to 17
Sand	50 to 62
Silt	25 to 32
Clay	2 to 6

5.5 Refusal

Auger refusal on probable boulders or bedrock was encountered below the silty sand at depths and elevations shown in Table 5.1.

Table 5.1 – Depths and Elevations to Auger Refusal

Location relative to existing culvert	Borehole	Depth (m)	Elevation (m)
North end	LRC-01	6.2	451.0
Middle	LRC-02	11.6	448.0
South end	LRC-03/DCPT	8.4	449.1

5.6 Water Levels

Water levels were observed in the open boreholes upon completion of the drilling operations.

Artesian conditions were encountered during drilling operations within the silty sand layer below 4.9 m and 5.5 m depth (elevations 452.3 and 452.0) in Boreholes LRC-01 and LRC-03. A standpipe piezometer was installed in Boreholes LRC-01 and LRC-03 to monitor water levels after completion of drilling.

The water levels measured in the open boreholes and piezometers are summarized in Table 5.2.

Table 5.2 – Water Level Measurements

Borehole	Date	Water Level (m)		Comment
		Depth	Elevation	
LRC-01	Sept. 26, 2011	1.8	455.4	Open borehole
	Dec. 1, 2011	0.4*	457.6	Piezometer
	Oct. 28, 2012	0.4* (frozen)	457.6	Piezometer
LRC-02	Sept. 17, 2011	3.3	456.3	Open borehole
LRC-03	Sept. 26, 2011	1.2	456.3	Open borehole
	Dec. 1, 2011	0.6*	458.1	Piezometer
	Oct. 28, 2012	1.1* (frozen)	458.6	Piezometer

*Indicates water level above ground surface, artesian conditions.

The piezometric readings reveal that the groundwater level is 0.4 m to 1.1 m above the original ground surface (elevations 457.6 to 458.6), indicating artesian conditions at this site.

The above values are short-term readings and seasonal fluctuations of the groundwater level are to be expected. In particular, the groundwater level may be at a higher elevation after the spring snowmelt or after periods of heavy rainfall.

General arrangement (GA) drawing indicates that the Little Rest Creek water level was measured at Elevation 456.8 m on June 28, 2011.

6 MISCELLANEOUS

Borehole locations were selected and marked in the field by Thurber Engineering Ltd. Upon completion of drilling, the borehole elevations and coordinates were established from a drawing provided by Genivar.

Thurber obtained utility clearances for the borehole locations prior to drilling.

Eastern Ontario Diamond Drilling Ltd. from Hawkesbury, Ontario supplied a truck mounted CME 75 drill rig and conducted the drilling, sampling and in-situ testing operations for Borehole LRC-02. OGS Drilling Inc. of Almonte, Ontario supplied a portable tripod mounted drilling set up and conducted the drilling, sampling and in-situ testing operations for Borehole LRC-01 and LRC-03.

The field program was supervised by Mr. George Azzopardi and Mr. Mubashar Tahir of Thurber.

Routine laboratory testing was carried out by Thurber Engineering Ltd.

Overall supervision of the field program was conducted by Mr. Mark Farrant, P.Eng. Interpretation of the data and preparation of this report were carried out by Ms. Lindsey Blaine, E.I.T. and Ms. R. Palomeque Reyna, P.Eng.

The report was reviewed by Dr. P.K. Chatterji, P.Eng. a Designated Principal Contact for MTO Foundations Projects.

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PART 2: ENGINEERING DISCUSSION AND RECOMMENDATIONS

7 INTRODUCTION

This report presents interpretation of the geotechnical data in the factual report and presents geotechnical recommendations for design of a new culvert to replace the existing culvert at Little Rest Creek east of Savanne, Ontario. The existing culvert carries Little Rest Creek under Highway 17 in the District of Thunder Bay, Ontario.

The existing highway is a two-lane paved road and crosses the creek on approach embankments about 3.0 m to 4.0 m high.

Currently two CSP culverts carry Little Rest Creek under Highway 17. The existing culverts are both 2.4 m in diameter. The length of the culverts is 24.4 m. The existing Highway 17 grade at the culvert location is near elevation 459.6.

The proposed culvert (as shown on the Preliminary General Arrangement dated March 6, 2012) consists of two parallel sheet pile walls supporting a slab consisting of precast concrete panels. The new structure will have a span of 10.7 m and a length of 22.0 m of which 18.0 m will be capped by precast panels. The underside of the cap panel is at approximate elevation 458.7.

Genivar indicated that the existing highway grade over the culvert will be raised approximately 80 mm.

The proposed culvert design and installation methods have been established through discussions between the Structural Designers and MTO's Northwest Region Office, taking into consideration alternative culvert types, environmental restrictions, roadway protection requirements and availability of construction materials.

The discussions and recommendations presented in this report are based on the factual data obtained during the course of the investigation. The plans and profiles used for preparation of this report were provided by Genivar.

8 STRUCTURE FOUNDATIONS

In general terms, the overburden encountered at this site along the toe of the highway embankment consist of topsoil overlying native layers of silty clay and silty sand. The topsoil is 0.6 m to 1.2 m thick. At the locations drilled, the highway embankment fill consists of 4.0 m dense to compact sand. The fill overlies layers of native silty clay and silty sand. The native silty clay is soft to firm in consistency and extends to depths ranging from 4.9 m to 7.5 m (elevations 452.0 to 452.3). The native silty sand is generally compact to very dense. Auger refusal on probable bedrock was contacted at depths ranging from 6.2 m to 11.6 m (elevations 448.0 to 451.0).

The piezometric readings reveal that the groundwater level is 0.4 m to 1.1 m above the original ground surface (elevations 457.6 to 458.6), indicating artesian conditions at this site. General arrangement (GA) drawing indicates that the Little Rest Creek water level was measured at Elevation 456.8 m on June 28, 2011.

Recommendations are provided for a sheet pile foundation supporting the precast cap panels.

The following foundation alternatives with the corresponding geotechnical design parameters are also presented in the event that the culvert design concept changes to an open footing culvert:

- Spread footings on native soils
- Augered Caissons (drilled shafts)
- Driven piles

A comparison of the foundation alternatives based on advantages and disadvantages of each one is included in Appendix D.

8.1 Steel Sheet Pile Walls

The preferred culvert replacement structure at this site consists of precast cap panels supported on steel sheet piles.

The GA drawing indicates that the underside of the pile cap is near elevation 458.5.

Based on the soil conditions, it is recommended that the sheet piles be driven to refusal. Driven steel sheet piles will develop resistance to vertical loads through frictional resistance along the sides of the piles within the native firm silty clay and loose to very dense silty sand and end bearing will also contribute to the load carrying capacity of the sheet piles.

Refusal to augering on possible boulders or bedrock was encountered at elevations ranging from 448.0 to 451.0 at the three boreholes drilled at this site. It is anticipated that the sheet piles will encounter refusal at elevations varying from 448.0 to 451.0.

Table 8.1 shows the factored Geotechnical Resistances at ULS (per metre width of sheet pile) and Geotechnical Resistances at SLS recommended for EZ-88, XZ-100 and JZ-127 sheet pile sections driven to refusal at elevations ranging from 451.0 to 448.0.

8.1 – Recommended Axial Resistances of Steel Sheet Piles

Sheet Pile Section	Estimated Sheet pile Length (m) Below underside of pile cap*	Approximate Pile Tip Elevation (m)	Factored ULS Resistance per meter width (kN)	SLS** Resistance (kN)
EZ-88	7.5 to 10.5	Expected to vary from 451.0 to 448.0	300	250
XZ-100			350	300
JZ-127			400	335

*Underside of pile cap at elevation 458.5

**The SLS values are based on a vertical pile settlement of 25 mm

Foundation loads at the sheet pile walls were provided by Genivar, and are as follows:

- 135 kN/m (SLS)
- 195 kN/m (ULS)

Pile installation should be in accordance with OPSS 903.

The appropriate pile driving note is “Sheet piles to be driven to refusal at elevations ranging from 451.0 to 448.0”. An additional note should be included to indicate that installation of permanent sheet pile walls by vibratory equipment is not permitted.

The single borehole drilled through the highway embankment encountered generally sand fill and no major obstructions were encountered in the fill in this borehole. However, it must be recognized that embankment fills are heterogeneous and may contain obstructions such as rockfill that will impede driving of sheet piles. If such obstructions are encountered within the highway embankment fill or in the embankment slope, such obstructions must be removed to facilitate driving of sheet piles.

It must be noted that auger refusal was observed at varying elevations in the boreholes. Sheet piles should be provided with sheet pile tip protector to minimize any tip damage. Design of the permanent sheet pile walls must consider environmental conditions such as road salts and fluctuating water levels that may cause corrosion and reduce the service life of the structure.

The lateral resistance of sheet piles may be computed using the lateral earth pressure distribution and parameters presented in Section 9.

The piezometric readings revealed artesian conditions at this site, with water level at 0.4 m to 0.6 m above ground surface. It is anticipated that the layer of clay above the silty sand layer will act as a seal around the sheet piles, minimizing the potential for upward flow around the sheet piles. If artesian water flow is noted during sheet pile installation, a granular filter, about 1.0 m thick, should be provided covering the ground surface around the sheet piles to allow any upward flow to drain without undergoing loss of fines. The CA should refer this issue to the design team for resolution.

The ground water levels both in front of and behind the sheet pile walls, including artesian pressure conditions should be considered for the sheet piles design.

For the open footing culvert option, the following foundation alternatives were considered: Spread footings on native soils, augered Caissons (drilled shafts) and driven H-piles.

Spread footings are not recommended at this site due to the low available geotechnical resistance and potential of settlement in the native silty clay below the sand fill and topsoil. Also, groundwater levels at the site are high, including artesian conditions. Unwatering/groundwater control of temporary excavation particularly below the creek level will be difficult for construction of footings.

Caissons are also not recommended at this site since construction of caissons through compact to dense saturated silty sand below water table will be difficult and require specialized construction techniques. Unwatering of the caisson would be impractical and attempts to do so might result in continued flow of fines into the caisson excavation.

These foundation options were therefore not developed further.

H-piles driven to refusal on probable bedrock or boulders is considered a feasible foundation alternative at this site.

8.2 Driven Steel H-Pile Foundations

The subsurface conditions at the site are considered suitable for the design of an open footing culvert supported on steel H-piles driven to refusal encountered below the dense to very dense silty sand at approximate Elevations ranging from 448.0 to 451.0 as indicated in Table 5.1.

8.2.1 Axial Resistance

Table 8.2 shows the axial, factored Geotechnical Resistances at Ultimate Limit States (ULS_f) and Geotechnical Resistance at Serviceability Limit States (SLS) recommended for

an HP 310x110 pile section when driven to refusal at elevations ranging from 448.0 to 451.0, are:

8.2 – Recommended Axial Resistances of an HP 310x110

Approximate Pile Tip Elevation (m)	Factored ULS Resistance (kN)	SLS Resistance (kN)
448.0 to 451	600	500

8.2.2 Pile Tips

If driven piles are selected, the tips of all piles should be fitted with pile tip protection from an approved manufacturer such as Titus Steel (Standard H-point) or approved equivalent.

8.2.3 Pile Installation

Pile installation should be in accordance with OPSS 903.

8.2.4 Pile Driving

Pile driving must be controlled by the Hiley Formula and an ultimate pile resistance to be specified by the designer in accordance with Clause 3.3.2 (b) Construction Stage of the Structural Manual. The Hiley formula need not be used until the piles are within 2.0 m of the design tip elevation. The appropriate pile driving note is “Piles to be driven in accordance with Standard SS 103-11 using an ultimate resistance of “R” kN per pile”. “R” must have a minimum value of twice the design load at ULS.

To facilitate pile installation, embankment fill through which piles will be driven must not contain oversize material, i.e. no particles exceeding 75 mm in size.

8.2.5 Artesian conditions

The piezometric readings revealed that the groundwater level is 0.4 m to 0.6 m above ground surface (elevations 457.6 to 458.1), indicating artesian conditions within the silty sand layer at this site. Artesian pressure has the potential to cause flow up the pile shaft, with accompanying loss of fines.

However, since there is a layer of clay above the silty sand layer, it is expected that this clay layer will seal around the pile, minimizing the potential for upward flow around each pile shaft. If artesian water flow is noted during pile installation, a granular filter, about 1.0 m thick, should be provided covering the piles to allow any upward flow to drain without undergoing loss of fines. If artesian flow is noted during pile installation, the CA should refer this issue to the design team for resolution.

8.2.6 Downdrag

Downdrag on the piles is not considered to be an issue at this site, since minimal highway grade raise is proposed, approximately 80 mm.

8.2.7 Lateral Resistance

For cohesive soils, the lateral resistance of the piles may be calculated as follows:

$$\begin{aligned}k_s &= 67 \cdot S_u / D \quad (\text{kN/m}^3) \\p_{ult} &= 9 \cdot S_u \text{ (kPa) at and below a depth of } 3 \cdot D \text{ (m) reduced to} \\&\quad \text{zero at the ground surface}\end{aligned}$$

where

$$\begin{aligned}D &= \text{pile width in metres} \\S_u &= \text{undrained shear strength (kPa)}\end{aligned}$$

For cohesionless soils, the lateral resistance of the piles may be calculated using a value for the coefficient of horizontal subgrade reaction (k_s) and ultimate lateral resistance (p_{ult}) as follows:

$$\begin{aligned}k_s &= n_h \cdot z / D \quad (\text{kN/m}^3) \\p_{ult} &= 3 \cdot \gamma \cdot z \cdot K_p \quad (\text{kPa})\end{aligned}$$

where

$$\begin{aligned}z &= \text{depth of embedment of pile in metres} \\D &= \text{pile width in metres} \\n_h &= \text{value from Table 8.3} \\\gamma &= \text{unit weight (Table 8.3)} \\K_p &= \text{passive earth pressure coefficient (Table 8.3)}\end{aligned}$$

The above equations and recommended parameters may be used to analyze the interaction between a pile and the surrounding soil. The lateral pressures obtained from the analysis should not exceed the ultimate lateral resistance.

The spring constant, K , for analysis may be obtained by the expression, $K = k_s \cdot L \cdot D$ (kN/m), where k_s is the coefficient of horizontal subgrade reaction (kN/m³), D is the pile width (m) and L is the length (m) of the pile segment or element used in the analysis. The ultimate lateral resistance on any one segment of pile, P_{ult} , may be obtained from the expression, $P_{ult} = p_{ult} \cdot L \cdot D$. This represents the ultimate load at which the pile fails and will not support any additional load at greater displacements. It is recommended, however,

that the total lateral resistance assumed in one pile be limited to no more than 120 kN at ULS and 35 kN at SLS. Parameters for lateral pile resistance are shown in Table 8.2.

Table 8.2 – Parameters for Lateral Pile Resistance

Location	Elevation	n_h (kN/m ³)	S_u kPa	K_p	Unit Weight (kN/m ³)	Soil Conditions
East and west walls	459.6 to 456.6	5,000	-	3.3	21	Sand, compact to dense (FILL)
	456.6 to 452.0	-	35	2.6	9*	Silty clay, very soft to firm
	452.0 to 449.1	6,500	-	3.5	11*	Silty sand, compact to very dense

*Buoyant unit weight below the water table.

For lateral soil/pile group interaction analysis, the modulus of subgrade reaction (k_s) may have to be reduced based on pile spacing.

Where a pile group is oriented *perpendicular* to the direction of loading, group action may be considered by reducing values for k_s by a reduction factor R as follows:

Pile Spacing Perpendicular to Direction of Loading	Horizontal Subgrade Reaction Reduction Factor, R
4 D*	1.00
1 D*	0.50

* D is the width of the pile, and spacing is measured centre to centre

Where a pile group is oriented *parallel* to the direction of loading, group action may be considered by reducing values for k_s by a reduction factor R as follows:

Pile Spacing Parallel to Direction of Loading	Horizontal Subgrade Reaction Reduction Factor, R
8 D	1.00
6 D	0.70
4 D	0.40
3 D	0.25

Intermediate values may be obtained by interpolation.

8.3 Proposed Foundation

It is understood that based on environmental considerations and cost of cast-in-place concrete, the preferred solution for culvert replacement at this site is precast cap panels founded on sheet piles. This is a feasible foundation alternative.

For an open footing culvert, H-pile foundations are a feasible alternative.

8.4 Frost Cover

The depth of frost penetration at this site is 2.5 m. The base of all footings and pile caps, if employed, must be provided with a minimum of 2.5 m of earth cover as protection against frost action.

9 CULVERT BACKFILL AND LATERAL EARTH PRESSURES

Culvert backfill should consist of free-draining granular material conforming to OPSS Granular A or Granular B Type II specifications. The existing highway embankment fill consists of sand with some gravel and is not considered susceptible to frost action.

Heavy compaction equipment should not be used adjacent to the sheet pile walls and roof of the culvert. Compaction should be carried out in accordance with OPSS 501. Backfill for the culvert should be placed and compacted in simultaneous equal lifts on both sides of the culvert, and the top of backfill elevation should be within 400 mm on both sides of the culvert at all times.

In general, earth pressures acting on the culvert walls may be assumed to impose a triangular distribution governed by the characteristics of the backfill. For a fully drained condition, the pressures should be computed in accordance with the CHBDC but generally are given by the expression:

$$p = K (\gamma h + q)$$

where: p = horizontal pressure on the wall at depth h (kPa)

K = earth pressure coefficient (see Tables 9.1)

γ = bulk unit weight of retained soil (see Table 9.1)

h = depth below top of fill where pressure is computed (m)

q = value of any surcharge (kPa)

Earth pressure coefficients for backfill to the culvert are dependent on the material used as backfill and the inclination of the ground surface behind the wall. Recommended unfactored values for a level ground surface are shown in Table 9.1. The at-rest coefficients should be employed for restrained culvert walls. Active pressures shall be used for any wingwalls or unrestrained walls.

If the ground surface behind the sheet pile walls is sloping, the earth pressure parameters will increase. Thurber should be contacted to provide revised earth pressure parameters for this condition.

The parameters in the table correspond to full mobilization of active and passive earth pressures, and require certain relative movements between the wall and adjacent soil to produce these conditions. The values to be used in design can be assessed from Figure C6.16 in the Commentary to the Canadian Highway Bridge Design Code.

Table 9.1 – Earth Pressure Coefficients (K) for Horizontal Ground Surface

Condition	Earth Pressure Coefficient (K)			
	OPSS Granular A or OPSS Granular B Type II $\phi = 35^\circ$, $\gamma = 22.8 \text{ kN/m}^3$	Existing Sand Fill or OPSS Granular B Type I $\phi = 32^\circ$, $\gamma = 21.2 \text{ kN/m}^3$	Native Silty Clay $\phi = 27^\circ$, $\gamma = 18 \text{ kN/m}^3$	Native Silty Sand $\phi = 32^\circ$, $\gamma = 21.2 \text{ kN/m}^3$
Active (Unrestrained Wall)	0.27	0.30	0.37	0.30
At rest (Restrained Wall)	0.43	0.47	0.55	0.47
Passive (Movement Towards Soil Mass)	3.7	3.3	2.7	3.3

For the at-rest condition, all soil above a horizontal surface behind the wall should be treated as a surcharge load.

In accordance with Clause 6.9.3 of the CHBDC, a compaction surcharge should be added. The magnitude should be 12 kPa at the top of fill and decreasing to 0 kPa at a depth of 2.0 m for Granular B Type I or at a depth of 1.7 m for Granular A or Granular B Type II.

The design of the culvert must incorporate measures such as weepholes or subdrains to permit drainage of the culvert backfill, or alternatively the culvert walls should be designed to withstand the potential build-up of hydrostatic pressures behind the walls.

Since proposed grade change at this site is only 80 mm, foundation settlement is not an issue.

10 EROSION CONTROL

Erosion protection should be provided along any section of embankment slope that may be in contact with stream flow. We understand that the exiting creek/stream channel is not to be disturbed by culvert replacement work.

A vegetation cover should be established on all other exposed earth surfaces to protect against surficial erosion, in general accordance with OPSS 804.

11 EXCAVATION AND GROUNDWATER CONTROL

All excavation must be carried out in accordance with the Occupational Health and Safety Act (OHSA). For the purposes of the OHSA, the sand fill forming the existing embankment may be classified as Type 3 soils. The silty clay and silty sand below the water table are Type 4 soils.

The piezometric readings reveal that the groundwater level is 0.4 m to 1.1 m above the original ground surface (elevations 457.6 to 458.6), indicating artesian conditions at this site. General

arrangement (GA) drawing indicates that the Little Rest Creek water level was measured at Elevation 456.8 m on June 28, 2011.

It is understood that measures such as creek diversions will not be permitted to avoid disturbance of the creek.

Based on the preliminary culvert design, excavation below the groundwater level to construct the new sheet pile wall is not anticipated.

For any temporary excavation, the Contractor must be prepared to control the groundwater and surface water to permit construction in the dry.

The design of the dewatering system that may be required is the responsibility of the Contractor and the Contract Documents must alert him to this responsibility.

The Contractor should also be prepared to pump from sumps to remove any remaining seepage water or surface water collecting in an excavation. Placement of concrete (if required) must be done in the dry. Unwatering must remain operational and effective until the culvert is installed and backfilled.

The excavation and backfilling for foundations must be carried out in accordance with OPSS 902.

12 SEISMIC CONSIDERATIONS

The following seismic parameters should be used for design:

- Velocity Related Seismic Zone 0
- Zonal Velocity Ratio 0.0
- Acceleration Related Seismic Zone 0
- Zonal Acceleration Ratio 0.0
- Peak Horizontal Acceleration 0.02

The soil profile type at this site has been classified as Type I. Therefore, according to Table 4.4 of the CHBDC, a Site Coefficient “S” (ground motion amplification factor) of 1.0 should be used in seismic design.

In accordance with Clause 4.6.4 of the CHBDC, retaining structures should be designed using active (K_{AE}) and passive (K_{PE}) earth pressure coefficients that incorporate the effects of earthquake loading. The coefficients of horizontal earth pressure for seismic loading presented in Table 12.1 may be used:

Table 12.1 – Earth Pressure Coefficients for Earthquake Loading

Condition	Earth Pressure Coefficient (K)			
	OPSS Granular A or OPSS Granular B Type II $\phi = 35^\circ, \gamma = 22.8 \text{ kN/m}^3$	Existing Sand Fill or OPSS Granular B Type I $\phi = 32^\circ, \gamma = 21.2 \text{ kN/m}^3$	Native Silty Clay $\phi = 27^\circ$ $\gamma = 18 \text{ kN/m}^3$	Native Silty Sand $\phi = 32^\circ$, $\gamma = 21.2 \text{ kN/m}^3$
Active (K_{AE})*	0.28	0.32	0.38	0.32
Passive (K_{PE})	3.7	3.2	2.6	3.2
At Rest (K_{OE})**	0.45	0.50	0.57	0.50

* After Mononobe and Okabe, passive case assumes a horizontal surface in front of the wall.

** After Woods

The site overlies soft to firm cohesive soils and loose to dense silty sand deposits over bedrock and a high water table. A review of the subsurface conditions indicates the site is not susceptible for liquefaction under current conditions.

The existing embankments are above the groundwater level and are not considered to be in danger of undergoing liquefaction.

13 ROADWAY PROTECTION

During the new culvert construction, temporary excavation of existing embankments will be required. The culvert construction will be done in stages in order to keep at least one highway lane operational. Roadway protection will be required to facilitate staging of removals and support the existing Highway 17 adjacent to the excavation.

Roadway protection should be provided in accordance with OPSS 539 and designed for Performance Level 2.

Conventional steel soldier pile wall with timber lagging is a suitable option to provide temporary support to the soils during excavation. Timber lagging boards should be installed as soon as the soil face is exposed and properly prepared.

The following parameters apply for design of the temporary shoring system.

γ	=	20 kN/m ³	(bulk unit weight)
γ_w	=	10 kN/m ³	(submerged unit weight under groundwater table)
K_a	=	0.30	(Active pressure coefficient for road embankment fill)
	=	0.37	(Active pressure coefficient for silty clay)
K_p	=	3.3	(Passive pressure coefficient for road embankment fill)
	=	2.7	(Passive pressure coefficient for silty clay)

$$h_w = 456.8 \quad (\text{Groundwater level})$$

The actual pressure distribution acting on the shoring system is a function of the construction sequence and the relative flexibility of the wall and these factors must be considered when designing the shoring system.

Temporary groundwater and surface water control measures will be required during construction.

The design of roadway protection should be the responsibility of the Contractor. All shoring systems should be designed by a Professional Engineer experienced in such designs, who will determine an appropriate support system.

14 CONSTRUCTION CONCERNS

Potential construction concerns include, but are not necessarily limited to:

- The single borehole drilled through the highway embankment encountered sand fill with gravel and did not encounter any obstruction. However, it must be noted that embankment fills are heterogeneous and may contain obstructions such as rockfill. There may be erosion protection on the side slopes of the highway embankment. If the sheet piles encounter such obstructions, they must be removed to facilitate driving of sheet piles.
- The possibility of variable pile lengths due to auger refusal encountered at depths ranging from 6.2 m to 11.6 m (elevations 451.0 to 448.0) across the site.
- If artesian groundwater flow is observed during sheet pile or H-pile driving, or any other construction activities, the contractor or QVE must immediately advise the CA. The CA should refer this issue to the design team.
- Roadway protection must be provided to maintain traffic during construction. Temporary shoring systems should be properly designed by a Professional Engineer experienced in such designs.
- Erosion protection should be provided to the embankment surfaces after construction.

15 CLOSURE

Engineering analysis and preparation of the report were carried out by Ms. R. Palomeque Reyna, P.Eng.

The report was reviewed by Dr. P.K. Chatterji, P.Eng. a Designated Principal Contact for MTO Foundations Projects.

THURBER ENGINEERING LTD.

Rocío Palomeque Reyna, P.Eng.
Geotechnical Engineer



P. K. Chatterji, P.Eng.
Review Principal



Appendix A

Record of Borehole Sheets

METRIC

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 (%)	
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	W _p W W _L	WATER CONTENT (%)					GR SA SI CL
								20 40 60 80 100							
457.2 0.0	TOPSOIL, organics Very Loose Dark Brown Moist to Wet (600mm)	[Pattern]	1	SS	1										
456.6 0.6	Silty CLAY, occasional roots Soft to Firm Grey	[Pattern]	2	SS	2										
		[Pattern]	3	SS	5										
		[Pattern]	4	SS	7									0 0 68 32	
		[Pattern]	5	SS	3										
		[Pattern]	6	SS	7										
452.3 4.9	Silty SAND, trace to some gravel, trace clay Dense Grey Moist to Wet	[Pattern]	7	SS	43									8 62 25 5	
	Occasional bedrock fragments	[Pattern]	8	SS	34										
451.0 6.2	END OF BOREHOLE AT 6.2m UPON REFUSAL ON PROBABLE BEDROCK OR BOULDER. WATER LEVEL OBSERVED AT 1.8m DURING DRILLING. Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe with a 1.52m slotted screen.	[Pattern]	9	SS	59/ 0.150										
WATER LEVEL READINGS: DATE DEPTH (m) ELEV. (m) Sep.26/11 1.8 455.4 Dec.01/11 0.4* 457.6 Oct.28/12 0.4* (Frozen) 457.6 * Above Ground Surface (Artesian Condition)															

ONTMT4S 0840.GPJ 11/5/12

+ 3, × 3: Numbers refer to Sensitivity

RECORD OF BOREHOLE No LRC-02

1 OF 2

METRIC

W.P. 6941-10-00 LOCATION N 5 425 727.4 E 702 896.6 Little Rest Creek Culvert ORIGINATED BY GA
HWY 17 BOREHOLE TYPE Hollow Stem Augers COMPILED BY AN
DATUM Geodetic DATE 2011.09.17 - 2011.09.17 CHECKED BY RPR

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				UNIT WEIGHT Y kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	20 40 60 80 100	20 40 60 80 100	20 40 60 80 100		
459.6													
0.0	SAND, some gravel to gravelly, trace silt and clay Dense to Compact Brown Moist (FILL)		1	SS	39		459						26 66 8 (SI+CL)
			2	SS	25		458						
			3	SS	18		457						
			4	SS	13		456						
			5	SS	15		455						
455.6							454						
4.0	Silty CLAY Firm Grey to Reddish Brown Wet		6	SS	6		453						
			7	SS	5		452						
			8	SS	8		451						
452.1							450						
7.5	Silty SAND, trace to some gravel, trace clay Loose to Compact Grey Wet		9	SS	30								17 50 30 3

Continued Next Page

+ 3, X 3: Numbers refer to
Sensitivity 20
15 10 5 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No LRC-02

2 OF 2

METRIC

W.P. 6941-10-00 LOCATION N 5 425 727.4 E 702 896.6 Little Rest Creek Culvert ORIGINATED BY GA
 HWY 17 BOREHOLE TYPE Hollow Stem Augers COMPILED BY AN
 DATUM Geodetic DATE 2011.09.17 - 2011.09.17 CHECKED BY RPR

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	20 40 60 80 100	20 40 60 80 100	20 40 60 80 100	20 40 60 80 100		
	Continued From Previous Page													
448.0	Silty SAND, trace to some gravel, occasional cobbles and boulders Very Dense Grey Wet		10	SS	50/ 0.00		449							
11.6	END OF BOREHOLE AT 11.6m UPON AUGER REFUSAL ON PROBABLE BOULDERS. WATER LEVEL OBSERVED AT 3.3m. BOREHOLE BACKFILLED WITH HOLEPLUG FROM 11.6m TO 0.3m, THEN SAND & GRAVEL TO SURFACE.													

+ 3 . X 3 : Numbers refer to
Sensitivity

20
15 10 5
(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No LRC-03

1 OF 2

METRIC

W.P. 6941-10-00 LOCATION N 5 425 709.5 E 702 894.2 Little Rest Creek Culvert ORIGINATED BY MAT
HWY 17 BOREHOLE TYPE Wash Boring COMPILED BY AN
DATUM Geodetic DATE 2011.09.26 - 2011.09.26 CHECKED BY RPR

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 (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa		WATER CONTENT (%)							
457.5	TOPSOIL, organics, some clay, occasional roots Very Loose Brown Moist		1	SS	0									GR SA SI CL			
456.3			2	SS	4			○ UNCONFINED + FIELD VANE									
1.2	Silty CLAY Firm to Soft Grey	3	SS	5					● QUICK TRIAXIAL × LAB VANE								
		4	SS	6													
		5	SS	5													
		6	SS	3													
		7	SS	4													
452.0	Silty SAND, trace to some gravel, trace clay Compact to Very Dense Grey Wet		8	SS	19					2					0 0 43 57		
9			SS	51	3												
10			SS	74													
11			SS	29													
449.6	End of sampling and start DCPT at 7.9m																
449.1																	
8.4	END OF BOREHOLE AT 8.4m UPON REFUSAL ON PROBABLE BEDROCK. WATER LEVEL OBSERVED AT 1.2m DURING DRILLING. Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe with a 1.52m slotted screen.																

Continued Next Page

+ 3, × 3: Numbers refer to Sensitivity 20 15 10 5 (%) STRAIN AT FAILURE

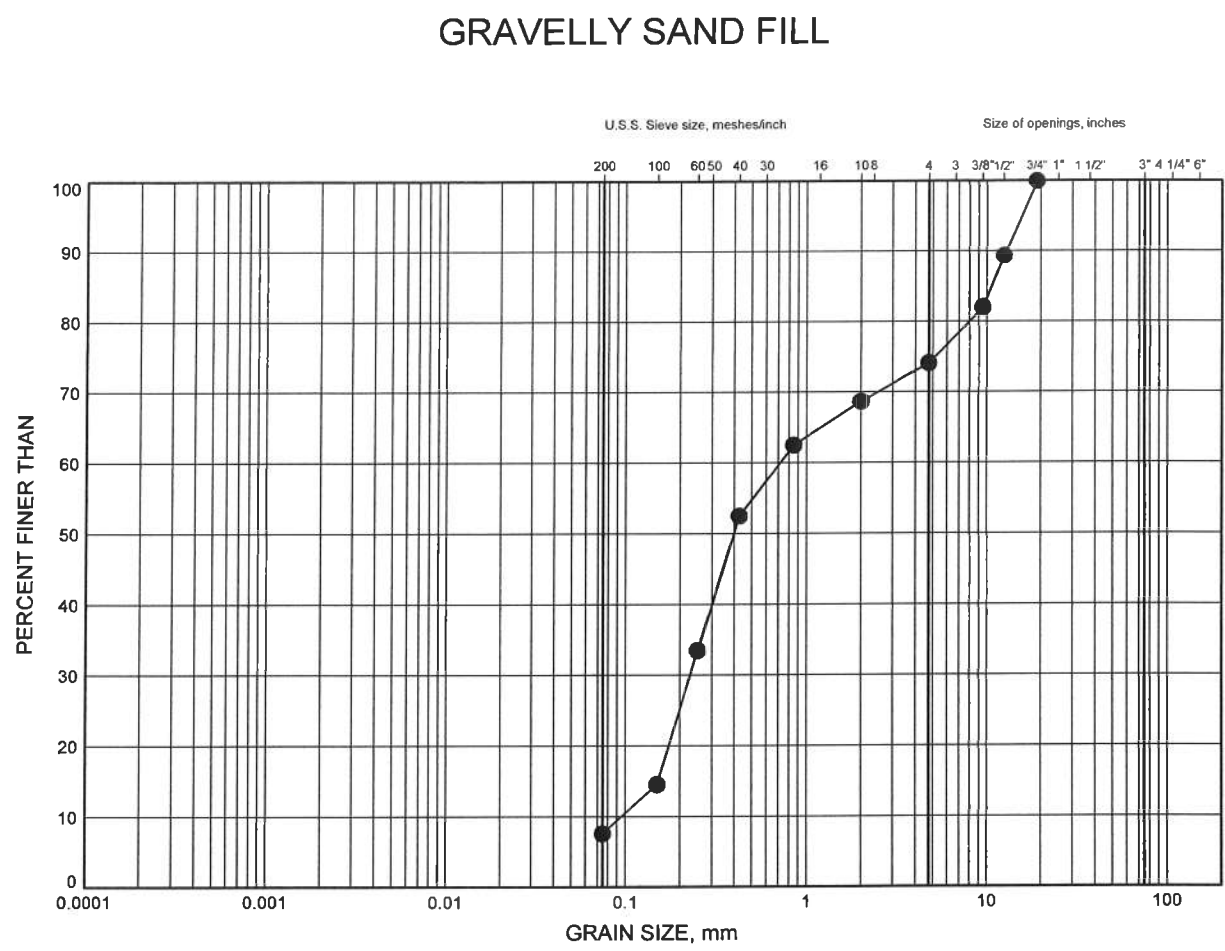
METRIC

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Appendix B
Laboratory Test Results

Little Rest Creek Culvert GRAIN SIZE DISTRIBUTION

FIGURE B1



SILT and CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
FINE GRAINED	SAND			GRAVEL		

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	LRC-02	1.07	458.53

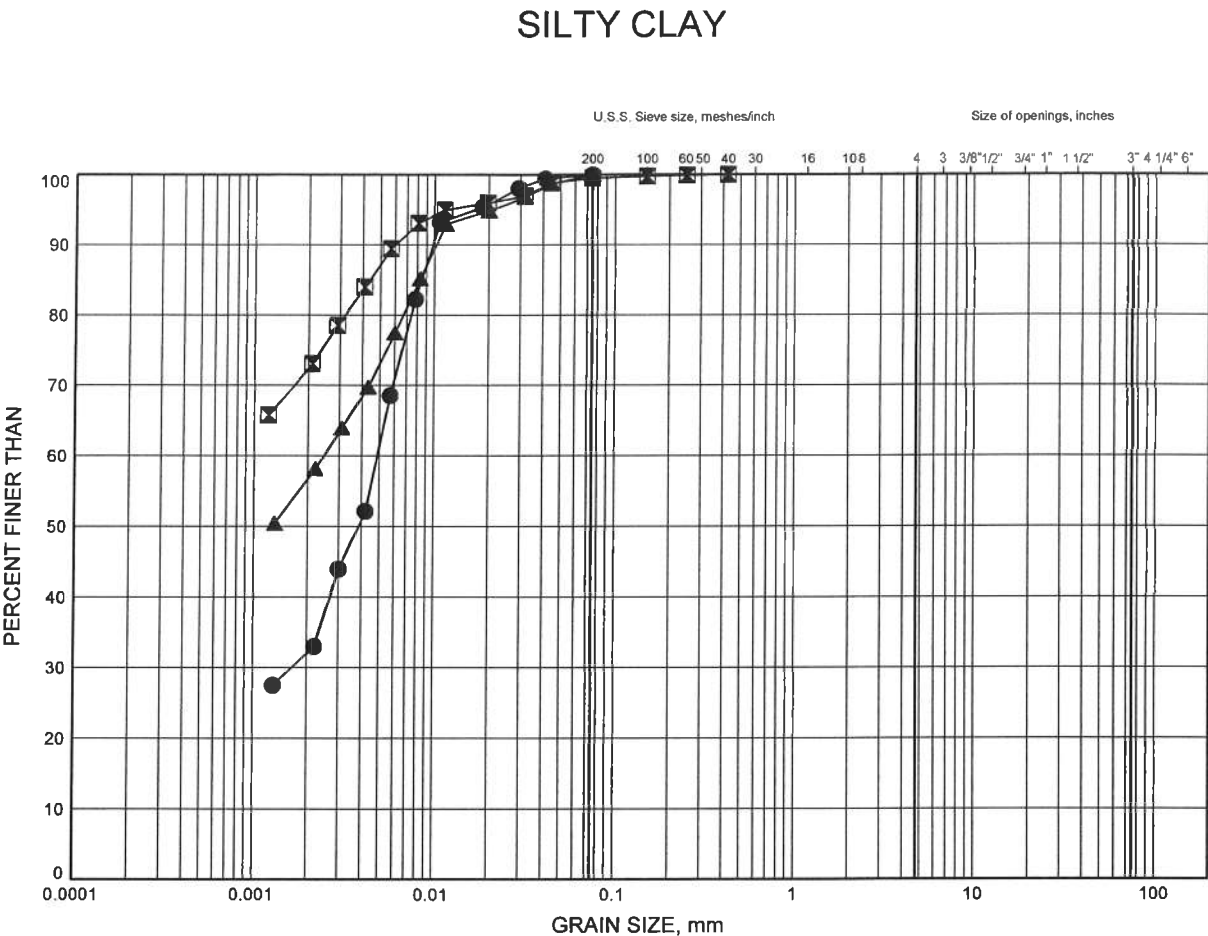


GRAIN SIZE DISTRIBUTION - THURBER 0840.GPJ 2/2/12

W.P.# 6941-10-00
 Prepared By AN
 Checked By LRB

Little Rest Creek Culvert GRAIN SIZE DISTRIBUTION

FIGURE B2



SILT and CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
FINE GRAINED	SAND			GRAVEL		

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	LRC-01	2.13	455.07
⊠	LRC-02	6.40	453.20
▲	LRC-03	3.96	453.54

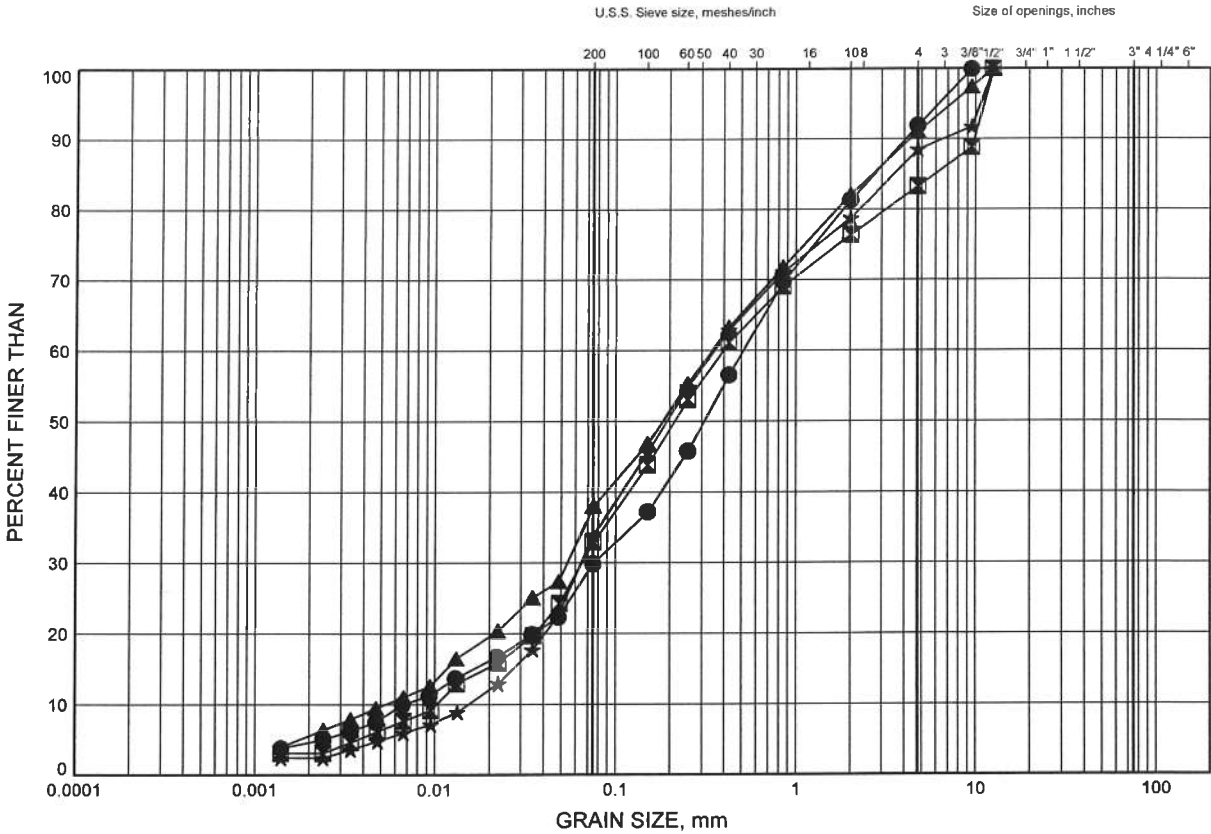


W.P.# 6941-10-00
 Prepared By AN
 Checked By LRB

Little Rest Creek Culvert GRAIN SIZE DISTRIBUTION

FIGURE B3

SILTY SAND



SILT and CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
FINE GRAINED	SAND			GRAVEL		

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	LRC-01	5.18	452.02
⊠	LRC-02	9.45	450.15
▲	LRC-03	5.79	451.71
★	LRC-03	7.01	450.49

GRAIN SIZE DISTRIBUTION - THURBER 0840.GPJ 2/2/12

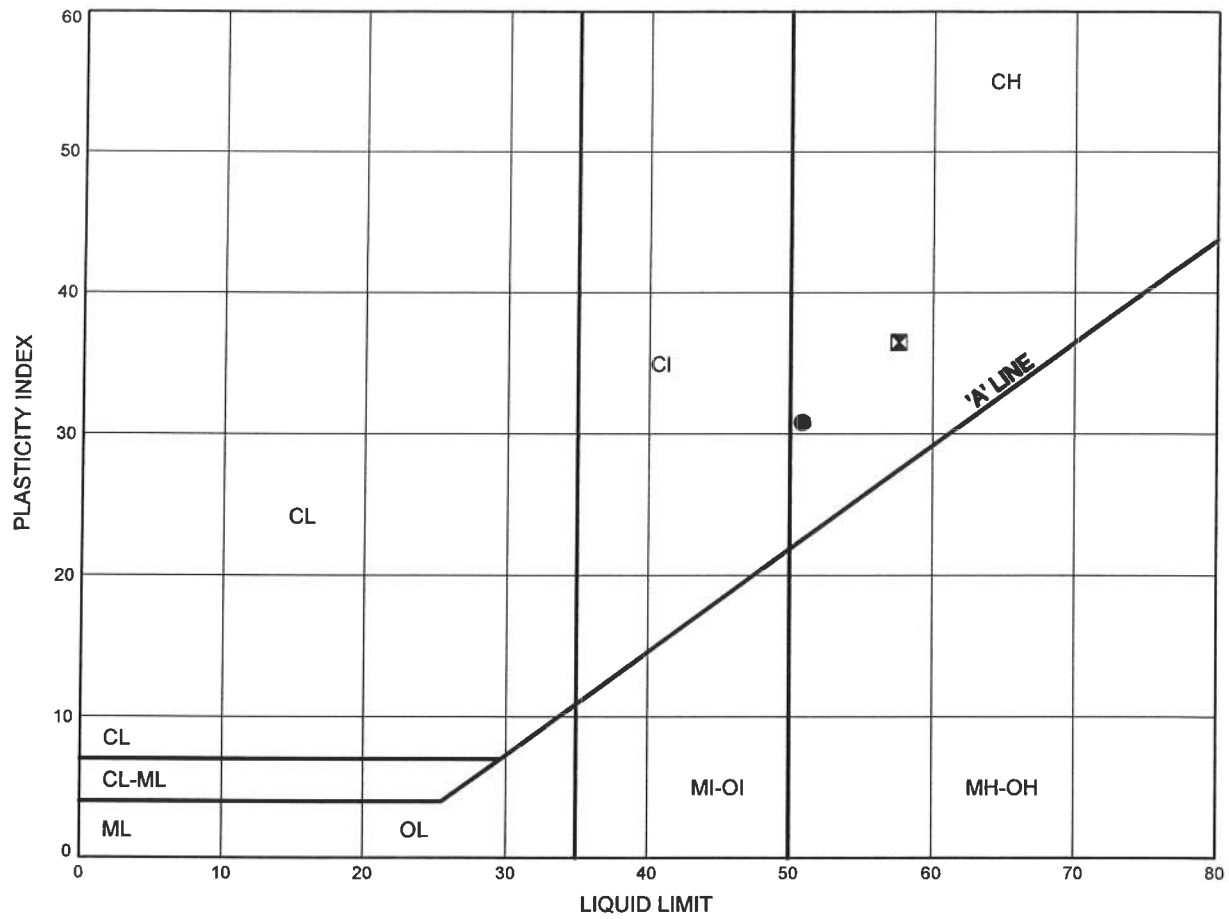
W.P.# .6941-10-00.....
 Prepared By .AN.....
 Checked By .LRB.....



Little Rest Creek Culvert
ATTERBERG LIMITS TEST RESULTS

FIGURE B4

SILTY CLAY



SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	LRC-02	6.40	453.20
⊠	LRC-03	3.96	453.54

Date March 2012
 Project 6941-10-00



Prep'd AN
 Chkd. LRB

Appendix C
Site Photographs



Photograph 1 – Highway 17 and Little Rest Creek culverts



Photograph 2 – North end of Little Rest Creek culverts, looking north



Photograph 3 – South end of Little Rest Creek culverts, looking towards Highway 17

Appendix D
Foundation Comparison

COMPARISON OF FOUNDATION ALTERNATIVES FOR EACH FOUNDATION ELEMENT

Preferred foundation		Open Footings Culvert Founded on		
Driven Sheet Piles	Driven H-Piles	Footings on Native Soil	Caissons	
<p>Advantages:</p> <ul style="list-style-type: none"> i. Minimizes potential for disturbance of streambed. ii. Ease of construction. iii. Provides shoring and foundation elements in one operation. iv. Installation of piles could continue in freezing weather. v. Potentially minimizes volume of excavation. <p>Disadvantages:</p> <ul style="list-style-type: none"> i. Unconventional design. ii. Cost of sheet piles. <p>FEASIBLE</p>	<p>Advantages:</p> <ul style="list-style-type: none"> i. Installation of piles could continue in freezing weather. ii. Foundation construction may require less volume of excavation than footings. iii. Readily installed. <p>Disadvantages:</p> <ul style="list-style-type: none"> i. Higher unit costs than footings. ii. Pile lengths required to reach refusal may vary. iii. Potential for artesian upward flow around pile shaft. <p>FEASIBLE</p>	<p>Advantages:</p> <ul style="list-style-type: none"> i. Generally less costly construction than deep foundation elements. <p>Disadvantages:</p> <ul style="list-style-type: none"> ii. Low available geotechnical resistance in native silty clay deposit. iii. Excavation to base of existing roadway embankment is required for footing construction. iv. High groundwater levels and artesian conditions. v. Dewatering will be required. vi. Potential disturbance of creek during excavation. <p>NOT RECOMMENDED</p>	<p>Advantages:</p> <ul style="list-style-type: none"> i. Construction of caissons could continue in freezing weather. ii. High geotechnical resistance available for units founded on refusal on probable bedrock. <p>Disadvantages:</p> <ul style="list-style-type: none"> iii. Higher cost than spread footings iv. Specialized installation measures such as temporary liners and drilling mud will be required to install caissons in artesian groundwater conditions. v. Potential difficulty in cleaning and inspecting bases. vi. Artesian conditions <p>NOT RECOMMENDED</p>	

Appendix E

List of SPs and OPSS, and Suggested Text for Selected NSSP

1. List of Special Provisions and OPSS Documents Referenced in this Report

- OPSS 903
- OPSS 501
- OPSS 804
- OPSS 902
- OPSS 539

2. Suggested Text for NSSP on sheet pile installation

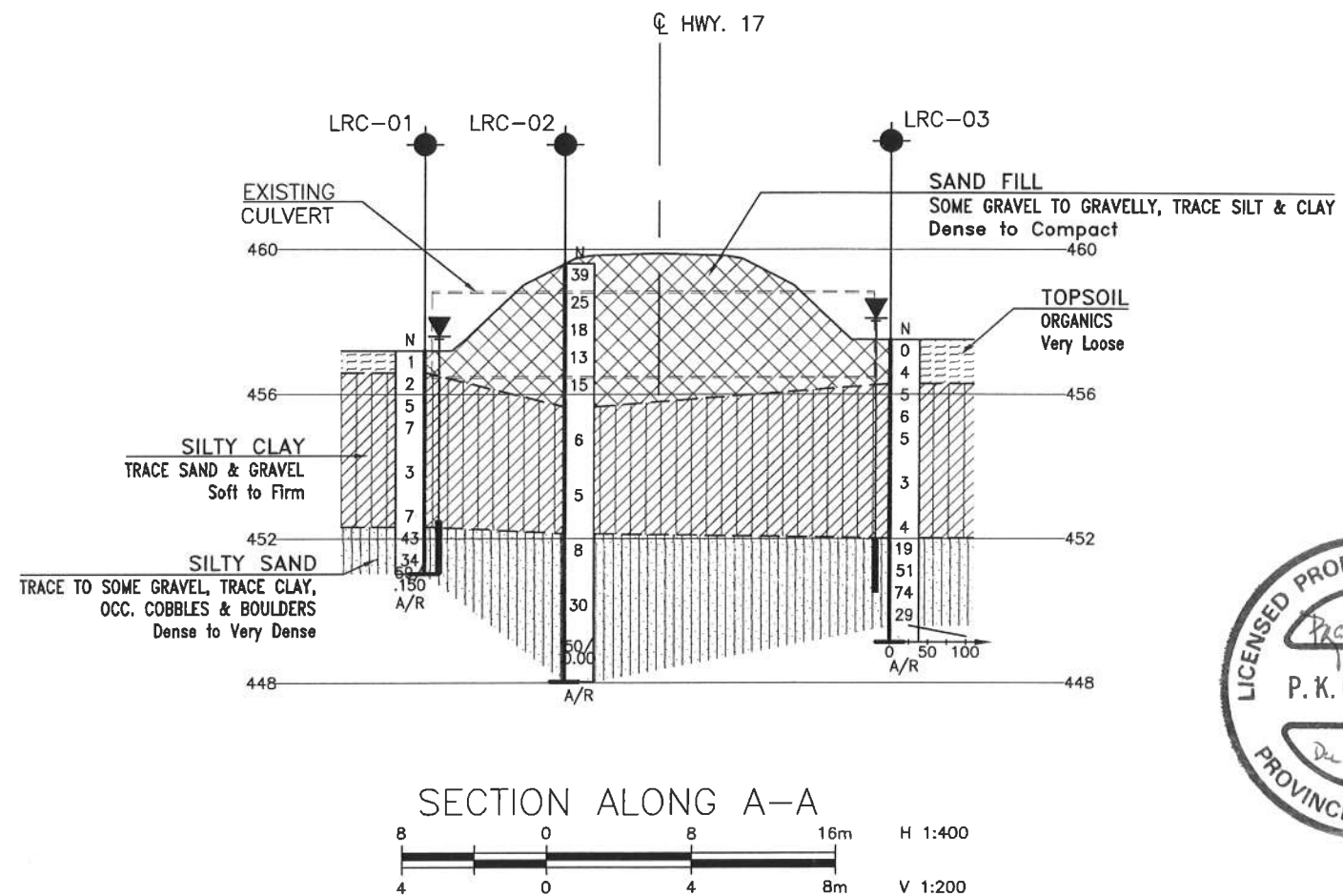
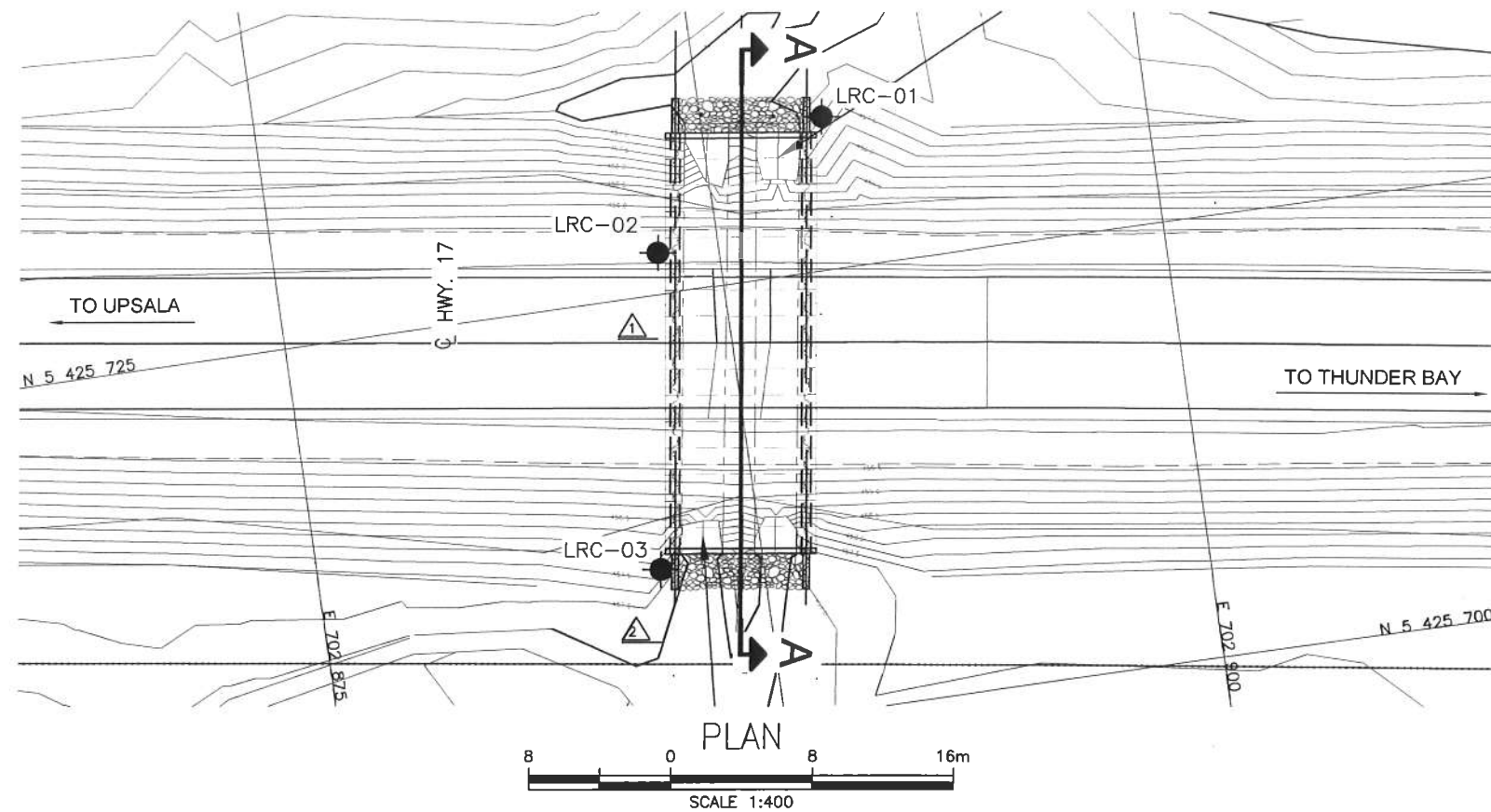
Vibratory equipment must not be used for sheet pile installation.

If a pile meets refusal at a depth less than the anticipated depth, the QVE must terminate driving before the pile is damaged due to over-driving

Appendix F

Drawing

Borehole Locations and Soil Strata



METRIC
DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN

CONT No
GWP No
WP No 6941-10-00



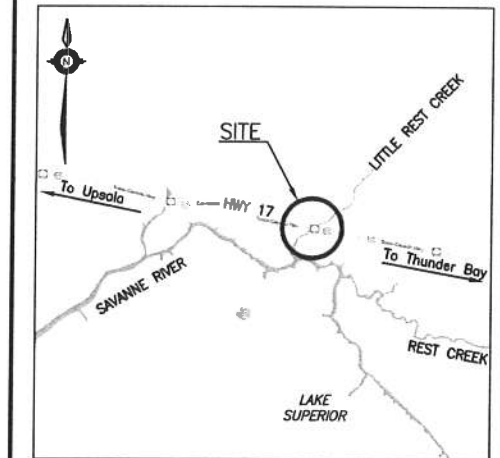
LITTLE REST CREEK
CULVERT REPLACEMENT
BOREHOLE LOCATIONS AND SOIL STRATA

SHEET

GENIVAR



THURBER ENGINEERING LTD.



KEYPLAN
LEGEND

◆	Borehole
◆	Borehole and Cone
N	Blows /0.3m (Std Pen Test, 475J/blow)
CONE	Blows /0.3m (60' Cone, 475J/blow)
PH	Pressure, Hydraulic
W	Water Level
HA	Head Artesian Water
P	Piezometer
90%	Rock Quality Designation (RQD)
A/R	Auger Refusal

NO	ELEVATION	NORTHING	EASTING
LRC-01	457.2	5 425 733.8	702 906.8
LRC-02	459.6	5 425 727.4	702 896.6
LRC-03	457.5	5 425 709.5	702 894.2

-NOTES-

- 1) The boundaries between soil strata have been established only at Borehole locations. Between Boreholes the boundaries are assumed from geological evidence.
- 2) This drawing is for subsurface information only. Surface details and features are for conceptual illustration.

GEOCREs No. 52B-15



REVISIONS	DATE	BY	DESCRIPTION
DESIGN	LRB	CHK	LRB
DRAWN	MFA	CHK	SITE
			STRUCT
			DWG 1