

**FOUNDATION INVESTIGATION AND DESIGN REPORT  
CLOUD RIVER CULVERT REPLACEMENT  
HIGHWAY 61  
CLOUD BAY COMMUNITY, NEEBING MUNICIPALITY  
DISTRICT OF THUNDER BAY, ONTARIO**

**G.W.P. 6936-10-00, SITE No. 48W/184C**

**Geocres Number: 52A-156**

**Report to**

**Hatch Mott MacDonald**

Thurber Engineering Ltd.  
2010 Winston Park Drive, Suite 103  
Oakville, Ontario  
L6H 5R7  
Phone: (905) 829 8666  
Fax: (905) 829 1166

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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 replacement culvert that will carry Highway 61 over the Cloud River in the Cloud Bay Community 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, a 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 Hatch Mott MacDonald, under the Ministry of Transportation Ontario (MTO) Agreement Number 6010-E-0010.

**2 SITE DESCRIPTION**

The Cloud River culvert is located on Highway 61, between Cloud River Road and Little Trout Bay Road in the Cloud Bay Community, Neebing Municipality in the District of Thunder Bay, Ontario. The site is approximately 40 km south of Thunder Bay, Ontario.

The existing Cloud River culvert is a one-cell concrete box culvert. The length and width of the culvert are 46.1 m and 6.1 m, respectively. The water flows through the culvert from west to east. Part of a previously abandoned culvert exists just to the south of the existing culvert. The length of the abandoned culvert has not been confirmed.



The surrounding lands are undeveloped and heavily treed. A few residential dwellings are located near the existing culvert on both sides of Highway 61.

Photographs in Appendix C show the general nature of the site and the existing culvert structure.

Photographs 7 and 8 in Appendix C indicate presence of rock fill on the side slopes of the highway embankments.

The region is characterized by Precambrian meta-volcanic and meta-sedimentary rocks intruded by later stage mafic dikes. At this site, the native soils primarily consist of silts and clays.

### **3 SITE INVESTIGATION AND FIELD TESTING**

The original scope of work consisted of drilling four boreholes along the existing culvert alignment, two boreholes on Highway 61 lanes/shoulder and two boreholes near the inlet and outlet of the existing culvert. However, due to existing site conditions (steep and heavily treed embankment slopes below the highway and presence of standing water near the embankment toes, it was not possible to drill the boreholes at the ends of the existing culvert. Therefore, only two boreholes were drilled at this site, both through the shoulders of Highway 61.

The site investigation and field testing for this project was carried out on October 13 to 16, 2011 and consisted of drilling and sampling two boreholes (identified as CLD-02 and CLC-03), the locations of which are shown on the attached Borehole Locations and Soil Strata Drawing in Appendix G.

Boreholes CLD-02 and CLD-03 were drilled and sampled to 26.5 m depth (elevations 180.3 to 181.0). A Dynamic Cone Penetration Test (DCPT) was conducted below borehole termination in each borehole. The DCPTs were terminated upon refusal at depths of 33.5 m and 33.8 m (elevations 173.3 and 173.7) in Boreholes CLD-02 and CLD-03, respectively.

Subsequently, on March 28 and 29, 2012, at the request of MTO and Hatch Mott McDonald, three more boreholes (numbered 01 to 03) were drilled at this site to investigate for obstructions in the ground that would indicate the presence of the abandoned culvert which could impede the installation of sheet piles. The three boreholes were drilled between the existing culvert and the abandoned culvert, 2.0 m south of the proposed south sheet pile wall. Boreholes 01 to 03 were augered to depths ranging from 12.2 m to 15.2 m (elevations 191.8 to 195.0). No sampling or laboratory testing were be conducted on the 3 boreholes.

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

Borehole locations were selected and established in the field by Thurber Engineering Ltd. Surveyors retained by Hatch Mott MacDonald provided site contour drawings from which the co-ordinates and the ground surface elevations for the boreholes were estimated. Boreholes 01 to 03 were marked on site by the surveyors.

Drilling on the highway shoulders was carried out using a truck-mounted CME 75 drill rig and the boreholes were advanced with hollow-stem augers and casing techniques. Samples were obtained at selected intervals using a split spoon sampler in conjunction with Standard Penetration Testing (SPT) in the embankment fill and native soils.

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 in the open boreholes were observed throughout the drilling operations. A standpipe piezometer consisting of 19 mm PVC pipe with slotted screen was installed in Borehole CLD-03 and enclosed in filter sand to permit longer term groundwater level monitoring. The boreholes were backfilled with bentonite holeplug in general accordance with O.Reg. 903 upon completion. The location and completion details of the piezometer and boreholes are shown in Table 3.1.

**Table 3.1 – Borehole Abandonment Details**

<b>Foundation Unit</b>	<b>Borehole</b>	<b>Piezometer Tip Depth/ Elevation (m)</b>	<b>Abandonment Details</b>
North wall	CLD-02	None installed	Borehole backfilled with holeplug to 3.7 m, sand and gravel to 0.1 m, then asphalt to surface.
South wall	CLD-03	25.5/182.0	Sand from 25.5 m to 22.0 m, holeplug from 22.0 m to 4.3 m, auger cuttings from 4.3 m to 0.3 m, sand from 0.3 m to 0.15 m, then asphalt to surface.

#### **4 LABORATORY TESTING**

All recovered soil samples were subjected to Visual Identification (VI) and natural moisture content determination. Selected samples were also subjected to grain size distribution analyses (sieve and hydrometer) and Atterberg Limits testing where appropriate. The results of this testing program are summarized on the Record of Borehole sheets included in Appendix A and on the figures presented 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 G. 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 soil stratigraphy encountered at this site consists of pavement structure overlying sand and gravel fill and silty clay fill. A layer of native sand and silt was contacted below the silty clay fill in one borehole. A native deposit of silt was contacted below the silty clay fill and the native sand and silt. Silty clay was encountered below the silt. DCPT were terminated upon refusal at 33.5 m and 33.8 m depth (elevations 174.0 and 173.7).

### **5.1 Pavement structure**

Pavement structure was encountered in the two boreholes drilled through the existing Highway 61 shoulders. The pavement structure in the shoulders consists of approximately 40 mm to 50 mm of asphalt overlying granular fill.

### **5.2 Sand Fill**

Brown sand fill containing some gravel was contacted below the pavement structure in both boreholes. The thickness of the fill was 1.3 m and 2.0 m in Boreholes CLD-02 and CLD-03, respectively.

The depths to the base of the sand fill were 1.3 m and 2.0 m (elevation 205.5).

SPT N-values recorded in the sand fill ranged from 6 to 21 blows per 0.3 m of penetration, indicating a loose to compact relative density.

The moisture content of the sand and gravel fill ranged from 5% to 19%.

As indicated earlier, rockfill is visible on the side slopes of the highway embankments. It is not confirmed if this rockfill is for erosion protection purposes or whether the existing embankment contains rockfill. No boreholes were drilled in these areas, where rockfill is visible in the sideslopes. It must be recognized that embankments fills are heterogeneous in nature and may contain obstructions such as boulders or rockfill.

### **5.3 Silty Clay Fill**

Reddish brown silty clay fill containing some sand to sandy and trace gravel and occasional cobbles was contacted below the sand fill in both boreholes. The thickness of the silty clay fill was in the two boreholes were 6.5 m and 7.7 m.

The depth to the base of the silty clay fill was 7.8 m and 9.7 m (elevations 199.0 and 197.8) in Boreholes CLD-02 and CLD-03, respectively.

SPT N-values recorded in the silty clay fill were 2 to 12 blows per 0.3 m of penetration, indicating a soft to stiff consistency.

The moisture content of the silty clay fill samples ranged from 20% to 41%.

Grain size distribution curves for selected silty clay fill samples are presented in Appendix B, Figure B1. The results are also summarized on the Record of Borehole sheets included

in Appendix A. Atterberg Limit test results are presented in Figures B5 of Appendix B. The results of the laboratory tests are summarized as follows:

<b>Soil Particles</b>	<b>Percentage (%)</b>
Gravel	1
Sand	14 to 25
Silt	37 to 41
Clay	33 to 48

<b>Index Property</b>	<b>Percentage (%)</b>
Liquid Limit	42 to 48
Plastic Limit	20 to 21

The above results show that the silty clay fill is of medium plasticity with a group symbol of CI.

#### **5.4 Sand and Silt**

Native grey sand and silt containing some clay, trace gravel and occasional wood fibres was contacted below the silty clay fill at 7.8 m depth (elevation 199.0) in Borehole CLD-02.

The depth to the base of the sand and silt was 9.4 m (elevation 197.4). The layer is 1.6 m in thickness.

SPT N-values recorded in the sand and silt were 11 and 21 blows per 0.3 m of penetration, indicating compact relative density.

The moisture content of the sand and silt was 36%.

Grain size distribution curve for a sand and silt sample is presented in Appendix B, Figure B2. The results are also summarized on the Record of Borehole sheets included in Appendix A. The results of the laboratory tests are summarized as follows:

<b>Soil Particles</b>	<b>Percentage (%)</b>
Gravel	3
Sand	44
Silt	43
Clay	10

#### **5.5 Silt**

Grey silt containing trace clay to clayey, trace sand, trace gravel and occasional cobbles and wood fibres was contacted below the native sand and silt layer at 9.4 m depth (elevation 197.4) in Borehole CLD-02 and below the silty clay fill at 9.7 m depth



(elevation 197.8) in Borehole CLD-03. The thickness of the silt layer was 11.6 m and 11.9 m in Boreholes CLD-02 and CLD-03, respectively.

The depth to the base of the silt layer was 21.0 m and 21.6 m (elevations 185.8 and 185.9) in Boreholes CLD-02 and CLD-03, respectively.

SPT N-values recorded in the silt ranged from 0 to 16 blows for 0.3 m of penetration, indicating a very loose to compact relative density.

The moisture content of samples collected from the silt layer generally varies between 21% and 36%.

Grain size distribution curves for selected silt samples are presented in Appendix B, Figure B3. The results are also summarized on the Record of Borehole sheets included in Appendix A. The results of the laboratory tests are summarized as follows:

Soil Particles	Percentage (%)
Gravel	0 to 1
Sand	0 to 9
Silt	79 to 92
Clay	8 to 21

## 5.6 Silty Clay

Native reddish brown silty clay containing trace sand was encountered below the silt at 21.0 m and 21.6 m depth (elevations 185.8 and 185.9) in both boreholes.

Boreholes CLD-02 and CLD-03 were both terminated within the silty clay layer at 26.5 m depth (elevations 181.0).

DCPTs were conducted below borehole termination depths and extended to refusal encountered at 33.5 m and 33.8 m depth (elevations 173.3 and 173.7) in Boreholes CLD-02 and CLD-03, respectively.

SPT 'N' values recorded in the silty clay ranged from 2 to 6 blows for 0.3 m of penetration, indicating a very soft to firm consistency.

The moisture content of samples collected from the silty clay layer generally varies between 21% and 43%.

Grain size distribution curves for selected silty clay samples are presented in Appendix B, Figures B4. The results are also summarized on the Record of Borehole sheets included in Appendix A. Atterberg Limits test results are presented in Figures B6 of Appendix B. The results of the laboratory tests are summarized as follows:

Soil Particles	Percentage (%)
Gravel	0
Sand	0
Silt	52
Clay	48

Index Property	Percentage (%)
Liquid Limit	43
Plastic Limit	19

The above results show that the silty clay is of medium plasticity with a group symbol of CI.

## 5.7 Water Levels

Water levels were observed in the boreholes during and upon completion of drilling. A standpipe piezometer was installed in Borehole CLD-03 to monitor water levels after completion of drilling. The water levels measured in the piezometer are summarized in Table 5.1, along with the measurements in the boreholes upon completion of drilling.

**Table 5.1 – Water Level Measurements**

Borehole	Date	Water Level		Comment
		Depth (m)	Elev. (m)	
CLD-02	October 16, 2011	8.2	198.6	Open borehole
CLD-03	November 30, 2011	4.8	202.7	Piezometer
	March 28, 2012	5.1	202.4	

Piezometric reading indicates that the water level at this site varies from 4.8 m to 5.1 m below the top of highway embankment, at elevations 202.7 to 202.4.

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.

Information provided by Hatch Mott MacDonald indicates that the water level along the edge of the river was near elevation 199.5 when the site was surveyed.

### **5.8 Boreholes drilled to investigate the presence of obstructions near proposed sheet piles**

Boreholes 01 to 03 were drilled on the south side of the proposed culvert, to determine if there are any obstructions that would indicate the presence of the abandoned culvert which could impede the installation of the sheet piles.

The borehole locations were positioned on site by surveyors approximately 2.0 m south of the proposed south sheet pile wall.

A review of the Boreholes 01 to 03 indicates that, with the exception of some grinding of drill augers at 1.8 m and 2.1 m depth in Boreholes 02 and 03, no major obstructions were noted during drilling the boreholes to depths of 12.2 m to 15.2 m (elevations 191.8 to 195.0.).

## **6 MISCELLANEOUS**

Borehole locations were selected and established in the field by Thurber Engineering Ltd. Surveyors retained by Hatch Mott MacDonald provided site contour drawings so that the co-ordinates and the ground surface elevations for the boreholes may be estimated from the contour drawings. Boreholes 01 to 03 were established on site by surveyors.

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.

The field program was supervised by Ms. Eckie Siu of Thurber.

Overall supervision of the field program was conducted by Mr. Mark Farrant, P. Eng. Interpretation of the data 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

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**PART 2: ENGINEERING DISCUSSION AND RECOMMENDATIONS**

**7 GENERAL**

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 the Highway 61 crossing of Cloud River in the Cloud Bay Community in the District of Thunder Bay, Ontario.

The existing Cloud River culvert is a one-cell concrete box culvert. The length and width of the culvert are 46.1 m and 6.1 m, respectively. The existing Highway 61 grade is near elevation 207.5. The existing highway embankments are approximately 8.0 m to 10.0 m high.

Part of a previously abandoned culvert exists just to the south of the existing culvert. The length of the abandoned culvert has not been confirmed.

The proposed replacement culvert (as shown on the Preliminary GA Drawing) will consist of two parallel sheet pile walls supporting a precast concrete panel cap. The new structure will have a span of approximately 9.4 m, and a length of 30.1 m of which 22.5 m will be capped by precast panels. The underside of the cap panel is at approximate elevation 205.2. The height of the sheet pile walls above the river bed will be in the order of 8.2 m.

The finished highway grade over the culvert will be maintained near elevation 207.5.

In addition to the proposed option of precast cap panels supported on sheet pile foundation, foundation recommendations are also provided for an open footing culvert if the design concept changes. The proposed culvert design and installation methods have been established through discussions between the structural engineer and MTO's Northwest Region office taking into consideration culvert types, environmental restrictions of working in or close to the river, roadway protection requirements and construction material availability. Alternative options such as box culvert, trenchless methods or culvert lining were excluded from further considerations.

The proposed culvert design was selected to avoid or minimize any disturbance or environmental impact on the river bed. The design also minimizes use of cast-in-place concrete which increases the cost of construction significantly.

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 Hatch Mott MacDonald.

## **8 STRUCTURE FOUNDATIONS**

In general, the soil stratigraphy encountered at this site consists of pavement structure overlying loose to compact sand fill and soft to stiff silty clay fill. The thickness of the highway embankment ranges from 7.8 m to 9.7 m. Below the embankment fill, a layer of native compact sand and silt was encountered in the borehole drilled at the location of the proposed north sheet pile wall. An extensive deposit of very loose to compact silt was encountered below the silty clay fill and native sand and silt layers. The thickness of the silt layer was 11.6 m and 11.9 m in the two boreholes. Silty clay was encountered below the silt. The silty clay was very soft to firm in consistency. DCPT were terminated upon refusal at 33.5 m and 33.8 m depth (elevations 173.3 and 173.7).

Measurements in the piezometer installed in Borehole CLD-03 during the current investigation, indicate that the groundwater level varies from 4.8 m to 5.1 m depth (elevations 202.7 to 202.4).

Information provided by Hatch Mott MacDonald indicates that the water level along the edge of the river was near elevation 199.5 when the site was surveyed.

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

Consideration was also given to the option of an open footing culvert supported on:

- Spread footings on native soils
- Augered Caissons (drilled shafts)
- Driven steel H-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. Driven steel sheet piles will develop resistance to vertical loads primarily through frictional resistance along the sides of the sheet piles within the native very loose to compact silt.

The factored Geotechnical Resistances at ULS (per metre width of sheet pile) and Geotechnical Resistances at SLS estimated for EZ-88 sheet pile sections driven to depths of 5, 10 and 15 m into the native silt are as follows:

**Table 8.1 – Recommended Axial Resistances of Steel Sheet Piles**

Sheet Pile Section	Embedment Length in Native Silt (m)	Approximate Pile Toe Elevation (m)	Factored ULS Resistance per meter width (kN/m)	SLS Resistance per meter width (kN/m)
EZ-88	5	192.0	170	140
	10	187.0	430	360
	15	182.0	770	640
XZ-100	5	192.0	190	160
	10	187.0	470	400
	15	182.0	850	710
JZ-127	5	192.0	200	170
	10	187.0	490	410
	15	182.0	880	735

The SLS values are based on a vertical pile settlement of 25 mm at the base of the embankment fill. Elastic compression of the pile above this level will be in addition to this settlement.

Pile installation should be in accordance with OPSS 903.

Sheet piles should be driven to the specified elevation noted in Table 8.1. The appropriate pile driving note is “Sheet piles to be driven to El. \_\_\_\_”. An additional note should be included to indicate that installation of permanent sheet pile walls by vibratory equipment is not permitted.

The depth of penetration of the sheet piles will be governed not only by the axial load, but also by the lateral pressure imposed by the soils retained behind the sheet piles.

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

Design of the permanent sheet pile walls must consider environmental conditions such as road salts or fluctuating water levels that may cause long term corrosion and reduce the service life of the structure.

Boreholes 01 to 03 were drilled between the existing culvert and the old abandoned culvert to determine if there would be any obstructions to install the south sheet pile wall. The boreholes indicate that with the exception of some grinding of drill augers at 1.8 m and

2.1 m depth in Boreholes 02 and 03, the boreholes did not encounter major obstructions within the depth of exploration of 12.2 m to 15.2 m (elevations 191.8 to 195.0).

It should however be noted that rockfill is visible on the side slopes of highway embankments. It is not confirmed whether this rockfill is for erosion protection purposes or the embankment contains rockfill. It must be recognized that embankment fills are heterogeneous in nature and may contain obstructions such as boulders or rockfill. If such obstructions are encountered at the proposed location of the sheet pile walls, they will have to be removed to facilitate driving of sheet piles.

For an open footing culvert, the following foundation options were considered:

## **8.2 Spread footings on native soils**

Consideration was given to supporting the culvert on spread footings founded on native soils, however this option is not recommended due to the following reasons:

1. Founding the spread footings on native subgrade soils will require a deep excavation through the fill, approximately 7.8 m to 9.7 m. Such an excavation would extend below the river level, would require extensive dewatering and yet would remain at risk of becoming destabilized due to the inflow of unbalanced groundwater heads.
2. A footing excavation will have environmental impact on the Cloud River.
3. The geotechnical resistance available in the native soils is relatively low and there is potential for settlement.
4. Spread footings could be subject to erosion or undermining/scour during high river flows.

In light of the above, the spread footings option was not further developed.

## **8.3 Augered Caissons (drilled shafts)**

Caissons are not recommended at this site since suitable end bearing materials were not encountered within the depth explored.

The DCPTs were terminated upon refusal at 33.5 m and 33.8 m depths (elevations 174.0 and 173.7). However, construction of caissons to these depths is not practical. The base of the caissons would be below the groundwater level, resulting in high hydrostatic heads at the base. Construction of caissons in very loose to compact saturated silts 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.



For these reasons, the use of a caisson foundation is not recommended.

#### 8.4 Driven Steel H-Piles

Driven steel H-piles will develop resistance to vertical loads primarily through frictional resistance along the shafts of the piles within the native very loose to compact silt.

The factored Geotechnical Resistances at ULS (per pile) and Geotechnical Resistance at SLS (25 mm settlement) estimated for an HP 310x110 H-pile section driven to various depths into the native silt are as follows:

**Table 8.2 – Recommended Axial Resistances for Steel H-Piles**

<b>Embedment Length in Native Silt (m)</b>	<b>Approximate Pile Tip Elevation (m)</b>	<b>Factored ULS Resistance per pile (kN)</b>	<b>SLS Resistance (kN)</b>
10	188.0	200	160
15	183.0	350	290

\*Native silt contacted near elevation 198.0

The structural resistance of the pile must be checked by the structural designer.

Pile installation should be in accordance with OPSS 903.

##### 8.4.1 Pile Lateral Resistance

The geotechnical lateral resistance acting on an H-pile in silt may be calculated using a value for the coefficient of horizontal subgrade reaction ( $k_s$ ) and ultimate lateral resistance ( $p_{ult}$ ) as follows:

$$k_s = n_h z / D \quad (\text{kN/m}^3)$$

$$p_{ult} = 3 \gamma z K_p \quad (\text{kPa})$$

where  $z$  = depth of embedment of pile in metres

$D$  = pile width in metres

$n_h$  = coefficient of horizontal subgrade reaction  
= 3,000 kN/m<sup>3</sup> in loose to compact silt below groundwater level

$\gamma$  = unit weight  
= 10 kN/m<sup>3</sup> (buoyant unit weight below water table)

$K_p$  = passive earth pressure coefficient  
= 2.9 for loose to compact silt

The ultimate lateral resistance of piles may be computed using the lateral earth pressure parameters presented in Section 9 with  $p_{ult} = K_p \gamma z$ . The coefficient of horizontal subgrade reaction may be computed using the equation above and a pile width  $D$  of unity.

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 must not exceed the ultimate lateral resistance.

The spring constant,  $K_s$ , for analysis may be obtained by the expression,  $K_s = k_s \times L \times D$  (kN/m), where  $k_s$  is the coefficient of horizontal subgrade reaction (kN/m<sup>3</sup>),  $D$  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,  $P_{ult}$ , may be obtained from the expression,  $P_{ult} = p_{ult} \times L \times D$ . 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.

The modulus of subgrade reaction may have to be reduced, based on the pile spacing. The reduction factors to be used for a pile group oriented perpendicular or parallel to the direction of loading are provided in Table 8.3. Intermediate values may be obtained by linear interpolation.

**Table 8.3 – Subgrade Reaction Reduction Factors for Pile Spacing**

Condition	Pile Spacing, Centre to Centre*	Reduction Factor
Pile group oriented <i>perpendicular</i> to direction of loading	4D	1.0
	1D	0.5
Pile group oriented <i>parallel</i> to direction of loading	8D	1.0
	6D	0.7
	4D	0.4
	3D	0.25

\* where  $D$  is the width of pile

Alternatively, horizontal loads may be resisted by means of battered piles.

#### 8.4.2 Downdrag

Downdrag on the piles is not considered to be an issue at this site, since no highway grade raise is proposed.

#### 8.5 Proposed Foundation

We understand 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.

Box culvert or an open footing culvert supported on H-pile foundations are also from a geotechnical perspective, feasible alternatives.

### **8.6 Frost Cover**

The design depth of frost penetration at this site is 2.2 m.

Frost protection should be provided for the undersides of all pile caps, if employed, and should consist of a minimum of 2.2 m of soil cover.

Based on the soil stratigraphy at Cloud River, frost treatment should be placed in accordance with OPSD 803.010.

## **9 CULVERT BACKFILL AND LATERAL EARTH PRESSURES**

Culvert backfill should consist of granular material conforming to OPSS Granular A or Granular B Type II specifications.

The sheet pile walls will be driven through the silty clay embankment fill into the native silt. The groundwater level is high at this site and is at elevation 202.4. Both the embankment fill and the underlying silt are susceptible to frost action and frost pressures will be exerted on the exposed portions of the sheet pile walls.

It is recommended that a minimum 2 m width of granular material be placed adjacent to the proposed sheet pile walls to provide drainage and protection against frost action in the frost-susceptible embankment fill and the underlying silt soils at the site. The granular material should extend 2.2 m (frost depth) below the exposed face of the sheet pile wall, or to the low water level in the river if above this depth. An alternative will be to provide insulation on the entire exposed portion of the sheet pile walls to minimize frost action. The insulation must be protected from damage.

If conventional culvert design is planned, the granular backfill should extend to the limits shown in OPSD 803.010.

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 Table 9.1)
$\gamma$	= 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)

Braced flexible sheeting typically undergoes insufficient deformation to establish a full active condition and a triangular pressure distribution may not apply. For the case of a strutted flexible wall, the lateral earth pressures should be computed using a trapezoidal distribution as shown on Figure 1 in Appendix E.

In a sheet pile wall with a single strut at the top of the wall (ie., the concrete cap), the triangular pressure distribution may underestimate the pressures near the top of the wall, as at-rest pressures may develop due to the restraint of the strut. However, we recommend that a design check be carried out to confirm the strut (cap) loads using the apparent pressure envelope for a single strut shown on Figure 2 in Appendix E, as per FHWA-IF-99-015, Geotechnical Engineering Circular No. 4, Ground anchors and anchored systems.

The design below the excavation base should be checked by comparing the active pressures to the passive resistance using both the triangular pressure distribution and apparent pressure envelope described above.

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 of the Commentary to the CHBDC.

**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 or Type III  $\phi = 32^\circ$ , $\gamma = 21.2 \text{ kN/m}^3$	Native Silty Clay and Silty Clay Fill  $\phi = 27^\circ$ , $\gamma = 18 \text{ kN/m}^3$	Native silt  $\phi = 29^\circ$ $\gamma = 20 \text{ kN/m}^3$
Active (Unrestrained Wall)	0.27	0.31	0.37	0.35
At rest (Restrained Wall)	0.43	0.47	0.55	0.51
Passive (Movement Towards Soil Mass)	3.7	3.3	2.7	2.9

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 no grade change is proposed at this site, foundation settlement is not an issue.

## 10 EROSION CONTROL

Erosion protection should be provided for the stream bed, culvert inlet/outlet areas at any embankment slope that may be affected by stream flow. Design of the erosion protection measures must consider hydrologic and hydraulic concerns and should be carried out by specialists experienced in this field.

Typically, rip-rap should be provided over all surfaces with which stream flow is likely to be in contact. Treatment should be in accordance with OPSS 810.010. A geotextile should be placed over the exposed subgrade prior to placement of the rip rap treatment in accordance with OPSS 511. 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 and silty clay fill forming the existing embankment may be classified as Type 3 soils. The silty clay fill and native silt below the water table are Type 4 soils.

The piezometric reading from the current investigation indicates that the groundwater level is at 4.8 m depth (elevation 202.7).

Information provided by Hatch Mott MacDonald indicates that the water level along the edge of the river was near elevation 199.5 when the site was surveyed.

We understand that measures such as stream diversions will not be permitted to avoid disturbance of the channel. The water level in the stream may be lower in the dry seasons and construction should be conducted during these periods.

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

Excavation below the groundwater level without prior dewatering is not recommended since the inflow of groundwater will cause boiling and sloughing of the soil below the water level making it difficult to maintain a dry, sound base on which to work. Prior excavation below the natural groundwater level, the groundwater must be depressed to a level at least 0.6 m below the deepest excavation level, sufficient to maintain a stable base and prevent soil disturbance by construction traffic.

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 and the need to engage a dewatering specialist.

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, November 2010.

## **12 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 the staging and support the existing Highway 61 adjacent to the excavation.

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

Continuous sheet pile wall or conventional steel soldier pile with timber lagging walls are two options that may be considered to provide temporary support to the embankments 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:

$\gamma$	=	21 kN/m <sup>3</sup>	(bulk unit weight)
$\gamma_w$	=	11 kN/m <sup>3</sup>	(submerged unit weight under groundwater table)
$K_a$	=	0.30	(Active pressure coefficient for road embankment sand fill)
	=	0.37	(Active pressure coefficient for silty clay fill)
$K_p$	=	3.0	(Passive pressure coefficient for road embankment sand fill)
	=	2.7	(Passive pressure coefficient for silty clay fill)
$h_w$	=	0	(assuming that the groundwater is maintained below the base of the excavation and that there is no hydrostatic pressure build-up behind a presumably permeable wall, soldier pile and lagging)
$h_w$	=	202.4	(elevation for hydrostatic pressure build-up behind sheet piles)

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 design of shoring with consideration of sufficient traffic loads and any sloping retained surface.

### 13 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 IV. Therefore, according to Table 4.4 of the CHBDC, a Site Coefficient “S” (ground motion amplification factor) of 2.0 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 13.1 may be used:

**Table 13.1 – 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 or Type III  $\phi = 32^\circ$ , $\gamma = 21.2 \text{ kN/m}^3$	Native Silty Clay and Silty Clay Fill  $\phi = 27^\circ$ , $\gamma = 18 \text{ kN/m}^3$	Native silt  $\phi = 29^\circ$ , $\gamma = 20 \text{ kN/m}^3$
Active ( $K_{AE}$ )*	0.28	0.32	0.38	0.36
Passive ( $K_{PE}$ )	3.7	3.2	2.6	2.9
At Rest ( $K_{OE}$ )**	0.45	0.50	0.57	0.54

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

\*\* After Woods

The site overlies loose to compact silts with high water table. A cursory review of the subsurface conditions indicates that a potential for liquefaction exists under the current conditions at this site.

Since the site is in a velocity related seismic zone of zero, this is not considered a significant risk. Localized liquefaction during a seismic event may result in local toe failure or a minimal embankment settlement which is expected to be readily repairable.



## 14 CONSTRUCTION CONCERNS

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

- 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 must be provided to the embankment surfaces after construction.
- Excavation below the water level, if required, will involve lowering of the groundwater level below the excavation base to maintain a reasonably dry excavation.
- Visual site inspection indicates presence of rockfill on the embankment side slopes. It is not confirmed whether this rockfill is for erosion protection or whether the embankment contains rockfill. It must be recognized that embankment fills are heterogeneous in nature and may contain obstructions such as boulders or rockfill. If the sheet piles encounter these conditions, the obstructions will have to be removed to facilitate driving of sheet piles.

## 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.

Rocio Palomeque Reyna, P.Eng., M.Eng.  
Geotechnical Engineer



Report reviewed by:  
P.K. Chatterji, P.Eng., Ph.D.  
Review Principal



## **Appendix A**

### **Record of Borehole Sheets**

## SYMBOLS, ABBREVIATIONS AND TERMS USED ON RECORDS OF BOREHOLES

### 1. TEXTURAL CLASSIFICATION OF SOILS

CLASSIFICATION	PARTICLE SIZE	VISUAL IDENTIFICATION
Boulders	Greater than 200mm	same
Cobbles	75 to 200mm	same
Gravel	4.75 to 75mm	5 to 75mm
Sand	0.075 to 4.75mm	Not visible particles to 5mm
Silt	0.002 to 0.075mm	Non-plastic particles, not visible to the naked eye
Clay	Less than 0.002mm	Plastic particles, not visible to the naked eye

### 2. COARSE GRAIN SOIL DESCRIPTION (50% greater than 0.075mm)

TERMINOLOGY	PROPORTION
Trace or Occasional	Less than 10%
Some	10 to 20%
Adjective (e.g. silty or sandy)	20 to 35%
And (e.g. sand and gravel)	35 to 50%

### 3. TERMS DESCRIBING CONSISTENCY (COHESIVE SOILS ONLY)

DESCRIPTIVE TERM	UNDRAINED SHEAR STRENGTH (kPa)	APPROXIMATE SPT <sup>(1)</sup> 'N' VALUE
Very Soft	12 or less	Less than 2
Soft	12 to 25	2 to 4
Firm	25 to 50	4 to 8
Stiff	50 to 100	8 to 15
Very Stiff	100 to 200	15 to 30
Hard	Greater than 200	Greater than 30

NOTE: Hierarchy of Soil Strength Prediction

- 1) Laboratory Triaxial Testing
- 2) Field Insitu Vane Testing
- 3) Laboratory Vane Testing
- 4) SPT value
- 5) Pocket Penetrometer

### 4. TERMS DESCRIBING DENSITY (COHESIONLESS SOILS ONLY)

DESCRIPTIVE TERM	SPT "N" VALUE
Very Loose	Less than 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very Dense	Greater than 50

### 5. LEGEND FOR RECORDS OF BOREHOLES

SYMBOLS AND ABBREVIATIONS FOR SAMPLE TYPE	SS Split Spoon Sample	WS Wash Sample	AS Auger (Grab) Sample
	TW Thin Wall Shelby Tube Sample	TP Thin Wall Piston Sample	
	PH Sampler Advanced by Hydraulic Pressure	PM Sampler Advanced by Manual Pressure	
	WH Sampler Advanced by Self Static Weight	RC Rock Core	SC Soil Core

$$\text{Sensitivity} = \frac{\text{Undisturbed Shear Strength}}{\text{Remoulded Shear Strength}}$$



Water Level

C<sub>pen</sub>


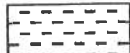



Shear Strength Determination by Pocket Penetrometer

- (1) SPT 'N' Value      Standard Penetration Test 'N' Value – refers to the number of blows from a 63.5kg hammer free falling a height of 0.76m to advance a standard 50 mm outside diameter split spoon sampler for 0.3 m depth into undisturbed ground.
- (2) DCPT              Dynamic Cone Penetration Test – Continuous penetration of a 50 mm outside diameter, 60° conical steel point attached to "A" size rods driven by a 63.5 kg hammer free falling a height of 0.76 m. The resistance to cone penetration is the number of hammer blows required for each 0.3 m advance of the conical point into undisturbed ground.

# UNIFIED SOILS CLASSIFICATION

MAJOR DIVISIONS		GROUP SYMBOL	TYPICAL DESCRIPTION
COARSE GRAINED SOILS	GRAVEL AND GRAVELLY SOILS	GW	Well-graded gravels or gravel-sand mixtures, little or no fines.
		GP	Poorly-graded gravels or gravel-sand mixtures, little or no fines.
		GM	Silty gravels, gravel-sand-silt mixtures.
		GC	Clayey gravels, gravel-sand-clay mixtures.
	SAND AND SANDY SOILS	SW	Well-graded sands or gravelly sands, little or no fines.
		SP	Poorly-graded sands or gravelly sands, little or no fines.
		SM	Silty sands, sand-silt mixtures.
		SC	Clayey sands, sand-clay mixtures.
FINE GRAINED SOILS	SILTS AND CLAYS $W_L < 50\%$	ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity.
		CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays. ( $W_L < 30\%$ ).
		CI	Inorganic clays of medium plasticity, silty clays. ( $30\% < W_L < 50\%$ ).
		OL	Organic silts and organic silty-clays of low plasticity.
	SILTS AND CLAYS $W_L > 50\%$	MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts.
		CH	Inorganic clays of high plasticity, fat clays.
		OH	Organic clays of medium to high plasticity, organic silts.
	HIGHLY ORGANIC SOILS		Pt
CLAY SHALE			
SANDSTONE			
SILTSTONE			
CLAYSTONE			
COAL			

## EXPLANATION OF ROCK LOGGING TERMS

ROCK WEATHERING CLASSIFICATION		SYMBOLS	
Fresh (FR)	No visible signs of weathering.		
Fresh Jointed (FJ)	Weathering limited to the surface of major discontinuities.		CLAYSTONE
Slightly Weathered (SW)	Penetrative weathering developed on open discontinuity surfaces, but only slight weathering of rock material.		SILTSTONE
Moderately Weathered (MW)	Weathering extends throughout the rock mass, but the rock material is not friable.		SANDSTONE
Highly Weathered (HW)	Weathering extends throughout the rock mass and the rock is partly friable.		COAL
Completely Weathered (CW)	Rock is wholly decomposed and in a friable condition, but the rock texture and structure are preserved.		Bedrock (general)

DISCONTINUITY SPACING		STRENGTH CLASSIFICATION			Field Estimation of Hardness*
Bedding	Bedding Plane Spacing	Rock Strength	Approximate Uniaxial Compressive Strength		
			(MPa)	(psi)	
Very thickly bedded	Greater than 2m	Extremely Strong	Greater than 250	Greater than 36,000	Specimen can only be chipped with a geological hammer
Thickly bedded	0.6 to 2m				
Medium bedded	0.2 to 0.6m	Very Strong	100-250	15,000 to 36,000	Requires many blows of geological hammer to break
Thinly bedded	60mm to 0.2m				
Very thinly bedded	20 to 60mm	Strong	50-100	7,500 to 15,000	Requires more than one blow of geological hammer to break
Laminated	6 to 20mm				
Thinly Laminated	Less than 6mm	Medium Strong	25.0 to 50.0	3,500 to 7,500	Breaks under single blow of geological hammer.
		Weak	5.0 to 25.0	750 to 3,500	Can be peeled by a pocket knife with difficulty
		Very Weak	1.0 to 5.0	150 to 750	Can be peeled by a pocket knife, crumbles under firm blows of geological pick.
		Extremely Weak (Rock)	0.25 to 1.0	35 to 150	Indented by thumbnail

TERMS	
Total Core Recovery: (TCR)	Core recovered as a percentage of total core run length.
Solid Core Recovery: (SCR)	Percent Ratio of solid core of full cylindrical shape recovered. Expressed with respect to the total length of core run.
Rock Quality Designation: (RQD)	Total length of sound core recovered in pieces 0.1m in length or larger as a percentage of total core run length.
Uniaxial Compressive Strength (UCS)	Axial stress required to break the specimen
Fracture Index: (FI)	Frequency of natural fractures per 0.3m of core run.

RECORD OF BOREHOLE No CLD-02

1 OF 4

METRIC

W.P. 6936-10-00 LOCATION Cloud River Culvert ORIGINATED BY ES  
HWY 61 BOREHOLE TYPE Hollow Stem Augers/Casing COMPILED BY AN  
DATUM Geodetic DATE 2011.10.15 - 2011.10.16 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 <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa		W F                      W                      W L					
206.8	ASPHALT: (40mm)						20   40   60   80   100								
205.5 1.3	SAND, some gravel Loose Brown Moist (FILL)		1	GS											
			1	SS	8										
			2	SS	11										
			3	SS	2										
			4	SS	2										
			5	SS	5										
199.0 7.8	SAND and SILT, some clay, trace gravel occasional wood fibres Compact Grey Wet		6	SS	3									1   14   37   48	
			7	SS	21										3   44   43   10
197.4 9.4	SILT, trace to some clay, trace sand, trace gravel, occasional cobbles and wood fibres		8	SS	11										

Continued Next Page

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

RECORD OF BOREHOLE No CLD-02

2 OF 4

METRIC

W.P. 6936-10-00 LOCATION Cloud River Culvert ORIGINATED BY ES  
HWY 61 BOREHOLE TYPE Hollow Stem Augers/Casing COMPILED BY AN  
DATUM Geodetic DATE 2011.10.15 - 2011.10.16 CHECKED BY RPR

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		FLASTIC LIMIT w <sub>p</sub>	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w <sub>L</sub>	UNIT WEIGHT  γ  kN/m <sup>3</sup>	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									

	Continued From Previous Page													
	SILT, trace to some clay, trace sand, trace gravel, occasional cobbles Very Loose to Loose Grey Wet		8	SS	4		196							
							195							
			10	SS	9		194							1 9 80 10
							193							
			11	SS	3		192							
							191							
			12	SS	9		190							0 0 89 11
							189							
	Compact to Loose		13	SS	16		188							
							187							
	Some clay to clayey		14	SS	9									

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





RECORD OF BOREHOLE No CLD-02

4 OF 4

METRIC

W.P. 6936-10-00 LOCATION Cloud River Culvert ORIGINATED BY ES  
HWY 61 BOREHOLE TYPE Hollow Stem Augers/Casing COMPILED BY AN  
DATUM Geodetic DATE 2011.10.15 - 2011.10.16 CHECKED BY RPR

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		FLASTIC LIMIT W <sub>P</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa	WATER CONTENT (%)					
	Continued From Previous Page							20 40 60 80 100						
173.3								20 40 60 80 100						
33.5	END OF DCPT AT 33.5m UPON REFUSAL. WATER LEVEL AT 8.2m UPON COMPLETION. BOREHOLE BACKFILLED WITH HOLEPLUG TO 3.7m, SAND AND GRAVEL TO 0.1m THEN ASPHALT TO SURFACE.							20 40 60 80 100						

# RECORD OF BOREHOLE No CLD-03

1 OF 4

METRIC

W.P. 6936-10-00 LOCATION Cloud River Culvert ORIGINATED BY ES  
 HWY 17 BOREHOLE TYPE Hollow Stem Augers COMPILED BY AN  
 DATUM Geodetic DATE 2011.10.13 - 2011.11.15 CHECKED BY RPR

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT w <sub>p</sub>	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w <sub>L</sub>	UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						
207.5								20 40 60 80 100						
207.5	ASPHALT: (50mm)													
	SAND, some gravel Compact to Loose Brown Damp (FILL)		1	GS			207							
			1	SS	21									
							206							
			2	SS	6									
205.5														
2.0	Silly CLAY, some sand to sandy, trace gravel Firm Reddish Brown (FILL)						205							1 25 41 33
			3	SS	4									
	Soft to Firm													
			4	SS	3		204							
							203							
			5	SS	5									
							202							
			6	SS	4		201							
							200							
	Stiff		7	SS	12		199							
			8	SS	9		198							
197.8														
9.7	SILT, trace to some clay													

Continued Next Page

+ 3 . X 3

Numbers refer to  
Sensitivity

20  
15  
10

(%) STRAIN AT FAILURE

## METRIC

[illegible]

+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to Sensitivity

# RECORD OF BOREHOLE No CLD-03

3 OF 4

METRIC

W.P. 6936-10-00 LOCATION Cloud River Culvert ORIGINATED BY ES  
 HWY 17 BOREHOLE TYPE Hollow Stem Augers COMPILED BY AN  
 DATUM Geodetic DATE 2011.10.13 - 2011.11.15 CHECKED BY RPR

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT w <sub>p</sub>	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w <sub>L</sub>	UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							WATER CONTENT (%)	
								○ UNCONFINED      + FIELD VANE								
						● QUICK TRIAXIAL      × LAB VANE										
Continued From Previous Page								20 40 60 80 100								
	SILT, some clay to clayey Compact Grey		15	SS	18									0 0 79 21		
							187									
185.9							186									
21.6	Silty CLAY Soft to Very Soft Reddish Brown															
			16	SS	4									0 0 52 48		
							184									
							183									
							182									
181.0			17	SS	2											
26.5	End of borehole and sampling at 26.5m Start DCPT at 26.5m						181									
							180									
							179									
							178									

Continued Next Page

+<sup>3</sup> · X<sup>3</sup> : Numbers refer to  
Sensitivity

20  
15  
10

(%) STRAIN AT FAILURE

# RECORD OF BOREHOLE No CLD-03

4 OF 4

METRIC

W.P. 6936-10-00 LOCATION Cloud River Culvert ORIGINATED BY ES  
 HWY 17 BOREHOLE TYPE Hollow Stem Augers COMPILED BY AN  
 DATUM Geodetic DATE 2011.10.13 - 2011.11.15 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 <sup>3</sup>	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	W <sub>p</sub> W W <sub>L</sub>	20 40 60			
	Continued From Previous Page													
173.7														
33.8	END OF DCPT AT 33.8m UPON REFUSAL Piezometer installation consists of 25mm diameter Schedule 40 PVC pipe with a 3.0m slotted screen.  WATER LEVEL READINGS: DATE DEPTH (m) ELEV. (m) Nov.30/11 4.8 202.7 Mar.28/12 5.1 202.4													

# RECORD OF BOREHOLE No BH-01

1 OF 2

METRIC

W.P. 6936-10-00 LOCATION N 5 330 939.8 E 345 610.3 Cloud River Culvert ORIGINATED BY RK  
 HWY 61 BOREHOLE TYPE Solid Stem Augers COMPILED BY AN  
 DATUM Geodetic DATE 2012.03.28 - 2012.03.29 CHECKED BY MEF

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC NATURAL LIQUID LIMIT MOISTURE LIMIT CONTENT			UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	20 40 60 80 100	w <sub>p</sub>	w	w <sub>L</sub>		
206.8														
0.0	ASPHALT: (125mm)													
0.1	SAND and GRAVEL Brown Moist (FILL)													
205.9							206							
0.9	Silty CLAY, trace to some sand, trace to some gravel Reddish Brown (FILL)													
			1	AS			205							
							204							
			1	SS	6									
	Firm						203							
	No sampling from 3.3m to 7.6m No obstructions noted/encountered						202							
							201							
							200							
198.9							199							
7.9	SAND and SILT, some clay, occasional wood fragments		2	SS	12									
198.6	Compact Grey Moist													
8.2	SILT, trace to some clay, trace sand, trace gravel Grey Wet						198							
							197							

Continued Next Page

+ 3. x 3. Numbers refer to 20 15 10 5 (%) STRAIN AT FAILURE  
 Sensitivity

## METRIC

[illegible]

RECORD OF BOREHOLE No BH-02

1 OF 2

METRIC

W.P. 6936-10-00 LOCATION N 5 330 939 1 E 345 611.8 Cloud River Culvert ORIGINATED BY RK  
 HWY 61 BOREHOLE TYPE Solid Stem Augers COMPILED BY AN  
 DATUM Geodetic DATE 2012.08.28 - 2012.08.29 CHECKED BY MEF

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				WATER CONTENT (%)			UNIT WEIGHT γ kN/m <sup>3</sup>	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	W <sub>P</sub>	W	W <sub>L</sub>				
207.0	ASPHALT: (100mm)						207									
0.0																
0.1	SAND and GRAVEL Brown Moist (FILL)															
206.1							206									
0.9	Auger probe to determine presence of obstructions (abandoned culvert) No sampling															
	Auger grinding at 1.8m						205									
							204									
	No obstructions noted/encountered						203									
							202									
							201									
							200									
							199									
							198									

Continued Next Page

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



RECORD OF BOREHOLE No BH-02

2 OF 2

METRIC

W.P. 6536-10-00 LOCATION N 5 330 939.1 E 345 611.8 Cloud River Culvert ORIGINATED BY RK  
HWY 61 BOREHOLE TYPE Solid Stem Augers COMPILED BY AN  
DATUM Geodetic DATE 2012.08.28 - 2012.08.29 CHECKED BY MEF

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT $\gamma$ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100		
	Continued From Previous Page							SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL X LAB VANE 20 40 60 80 100						
	No sampling No obstructions noted/encountered						197							
							196							
							195							
							194							
							193							
191.8							192							
15.2	END OF BOREHOLE AT 15.2m. BOREHOLE BACKFILLED WITH BENTONITE HOLEPLUG TO 0.6m, DRY CONCRETE TO 0.15m THEN COLD PATCH TO SURFACE.													

+<sup>3</sup>, X<sup>3</sup>; Numbers refer to  
Sensitivity 20  
15 5  
10 (%) STRAIN AT FAILURE

## METRIC

[illegible]

+ 3, × 3: Numbers refer to Sensitivity

RECORD OF BOREHOLE No BH-03

2 OF 2

METRIC

W.P. 6936-10-00 LOCATION N 5 330 938.0 E 345 613.7 Cloud River Culvert ORIGINATED BY RK  
HWY 61 BOREHOLE TYPE Solid Stem Augers COMPILED BY AN  
DATUM Geodetic DATE 2012.03.26 - 2012.03.29 CHECKED BY RPR

SOIL PROFILE				SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT w <sub>p</sub>	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w <sub>L</sub>	UNIT WEIGHT  γ  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)  GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	20 40 60 80 100			SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE								
	Continued From Previous Page																
	No sampling No obstructions noted/encountered						197										
							196										
195.0																	
12.2	END OF BOREHOLE AT 12.2m. BOREHOLE BACKFILLED WITH BENTONITE HOLEPLUG TO 0.6m, DRY CONCRETE TO 0.15m THEN COLD PATCH TO SURFACE.																

+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to  
Sensitivity

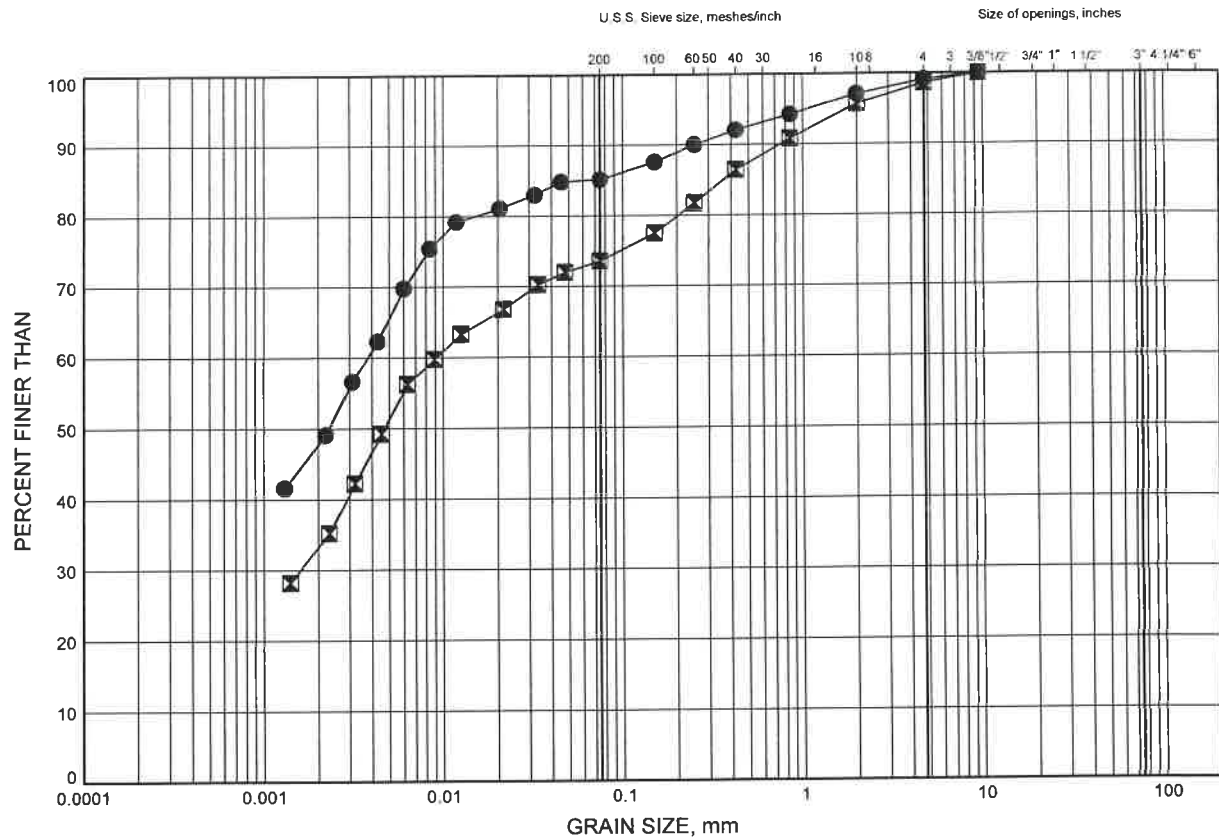
20  
15  
10  
5  
(%) STRAIN AT FAILURE

**Appendix B**  
**Laboratory Test Results**

# Cloud River Culvert GRAIN SIZE DISTRIBUTION

FIGURE B1

## SILTY CLAY FILL



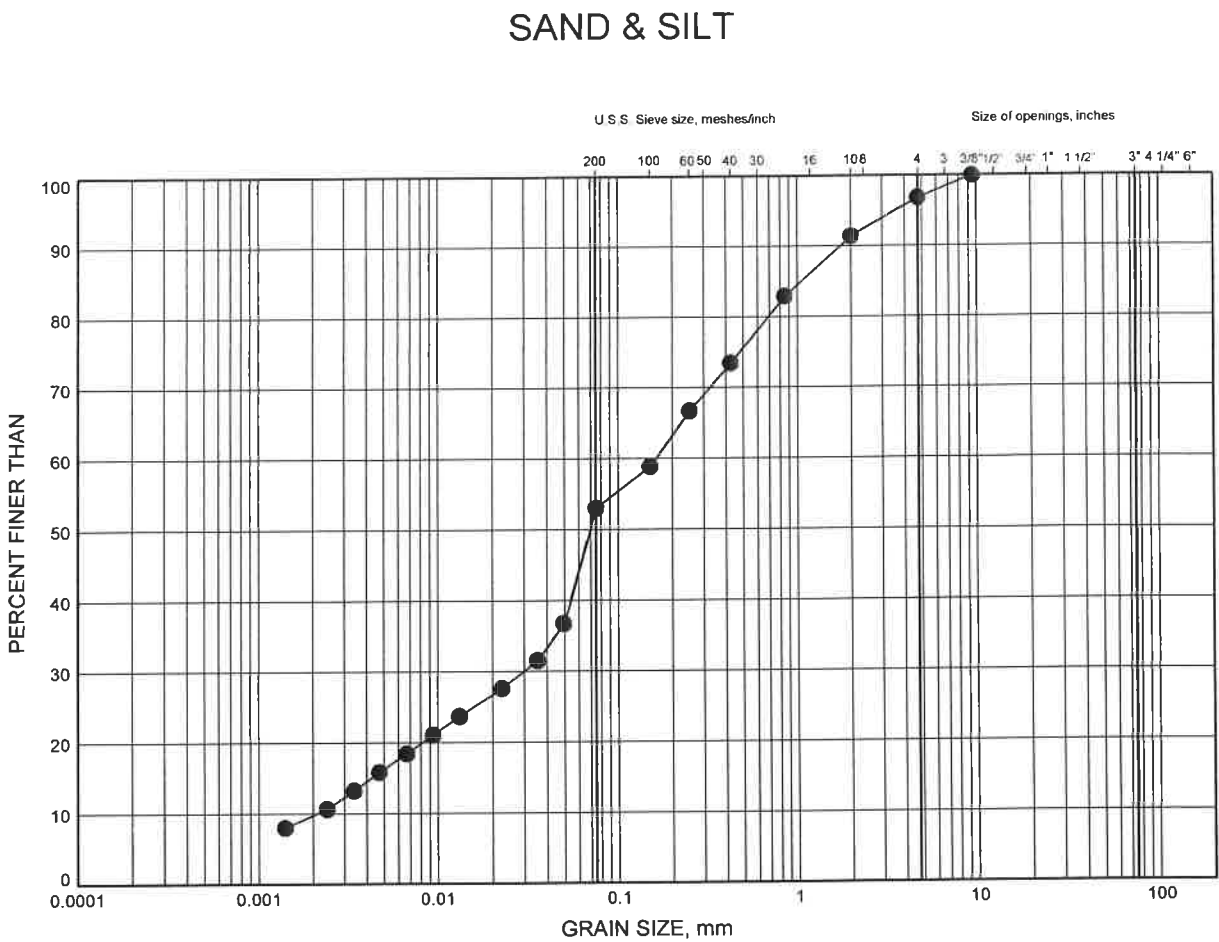
SILT and CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
FINE GRAINED	SAND			GRAVEL		

### LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	CLD-02	6.40	200.40
■	CLD-03	2.59	204.91

Cloud River Culvert  
GRAIN SIZE DISTRIBUTION

FIGURE B2



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	CLD-02	7.92	198.88

GRAIN SIZE DISTRIBUTION - THURBER 5121 GPJ 6/1/12

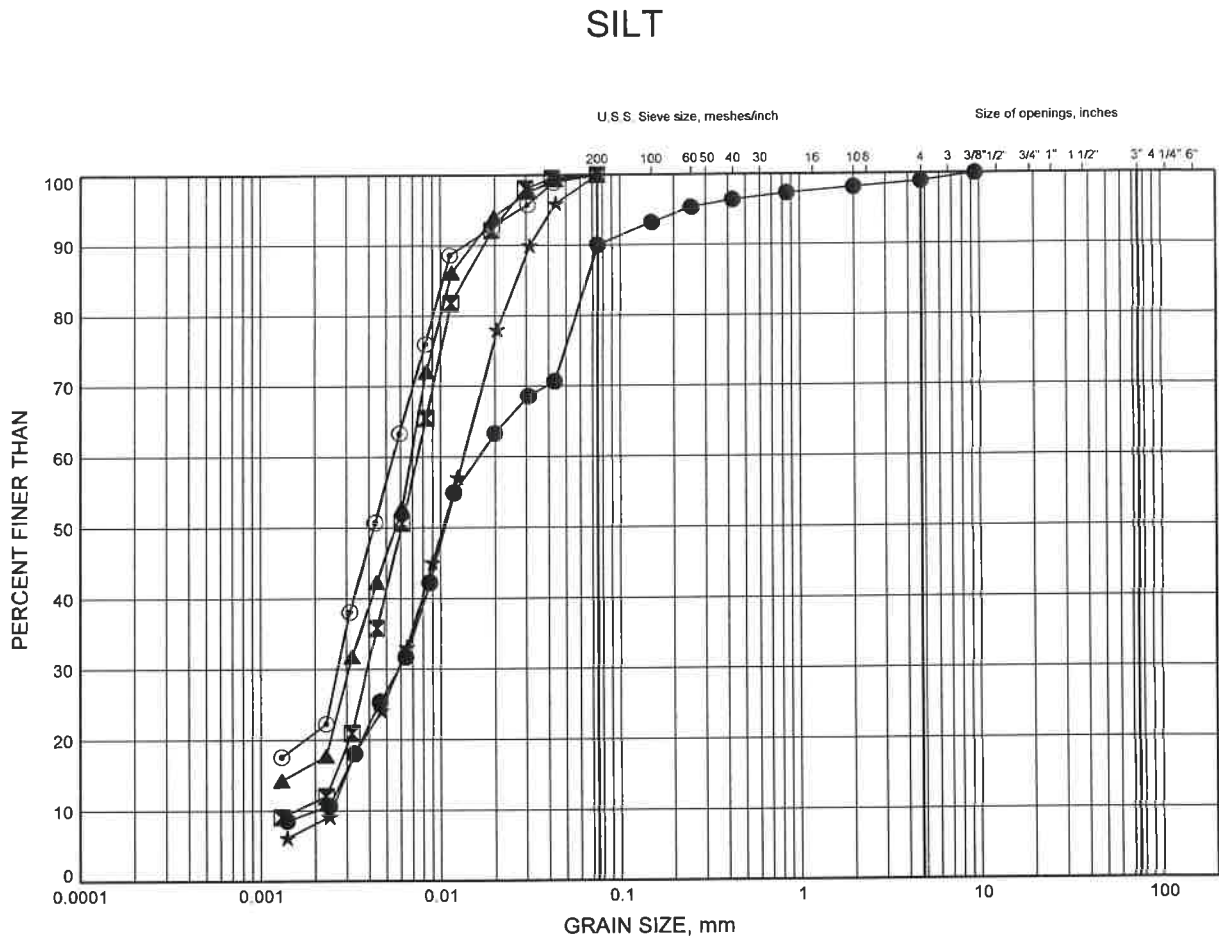
W.P.# 6936-10-00  
Prepared By AN  
Checked By RPR



THURBER

# Cloud River Culvert GRAIN SIZE DISTRIBUTION

FIGURE B3



## LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	CLD-02	12.50	194.30
⊠	CLD-02	15.54	191.26
▲	CLD-02	20.12	186.68
★	CLD-03	14.02	193.48
⊙	CLD-03	20.12	187.38

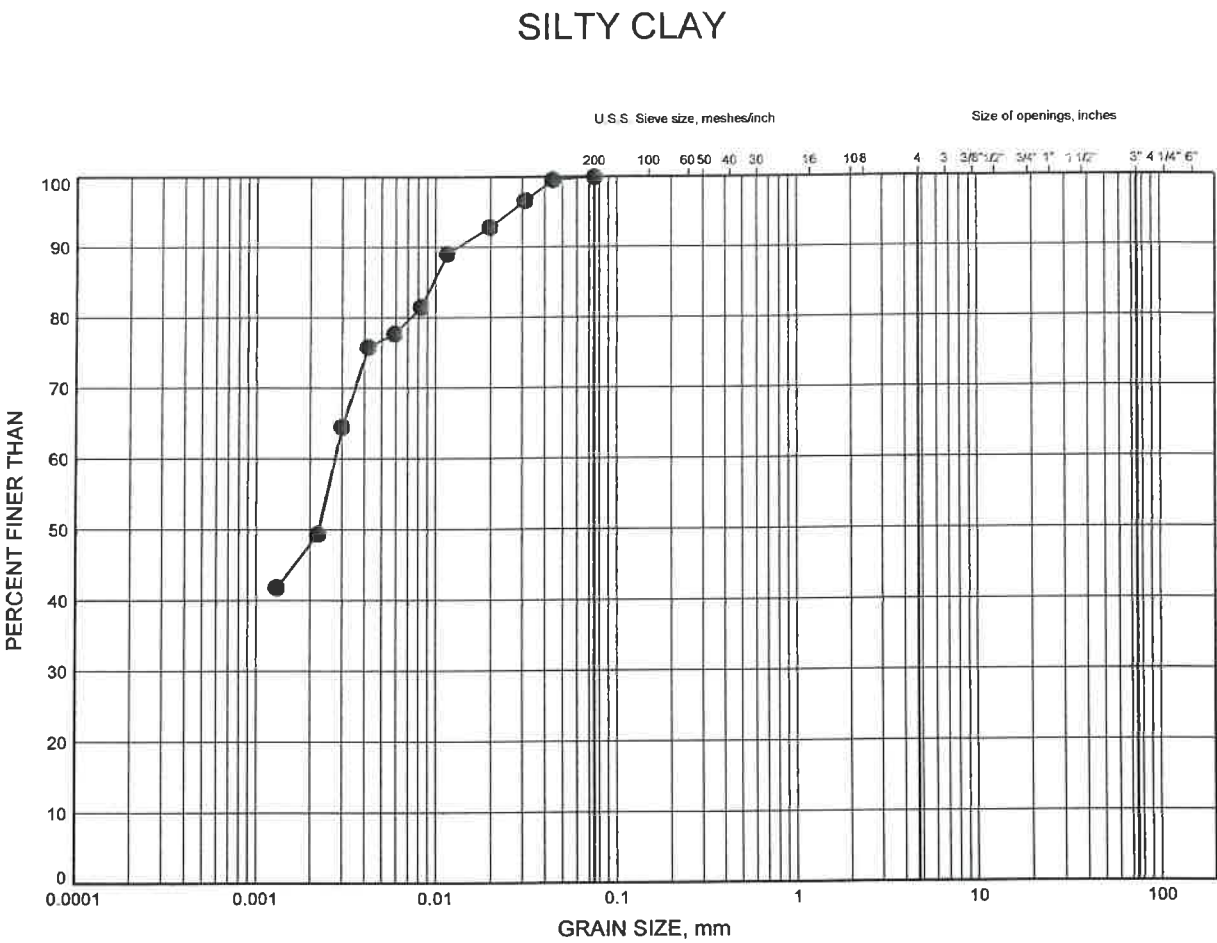
GRAIN SIZE DISTRIBUTION - THURBER 5121.GPJ 6/1/12

W.P.# 6936-10-00  
 Prepared By AN  
 Checked By RPR



# Cloud River Culvert GRAIN SIZE DISTRIBUTION

FIGURE B4



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

## LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	CLD-03	23.16	184.34

GRAIN SIZE DISTRIBUTION - THURBER 5121.GPJ 2/22/12

W.P.# 6936-10-00  
 Prepared By AN  
 Checked By RPR

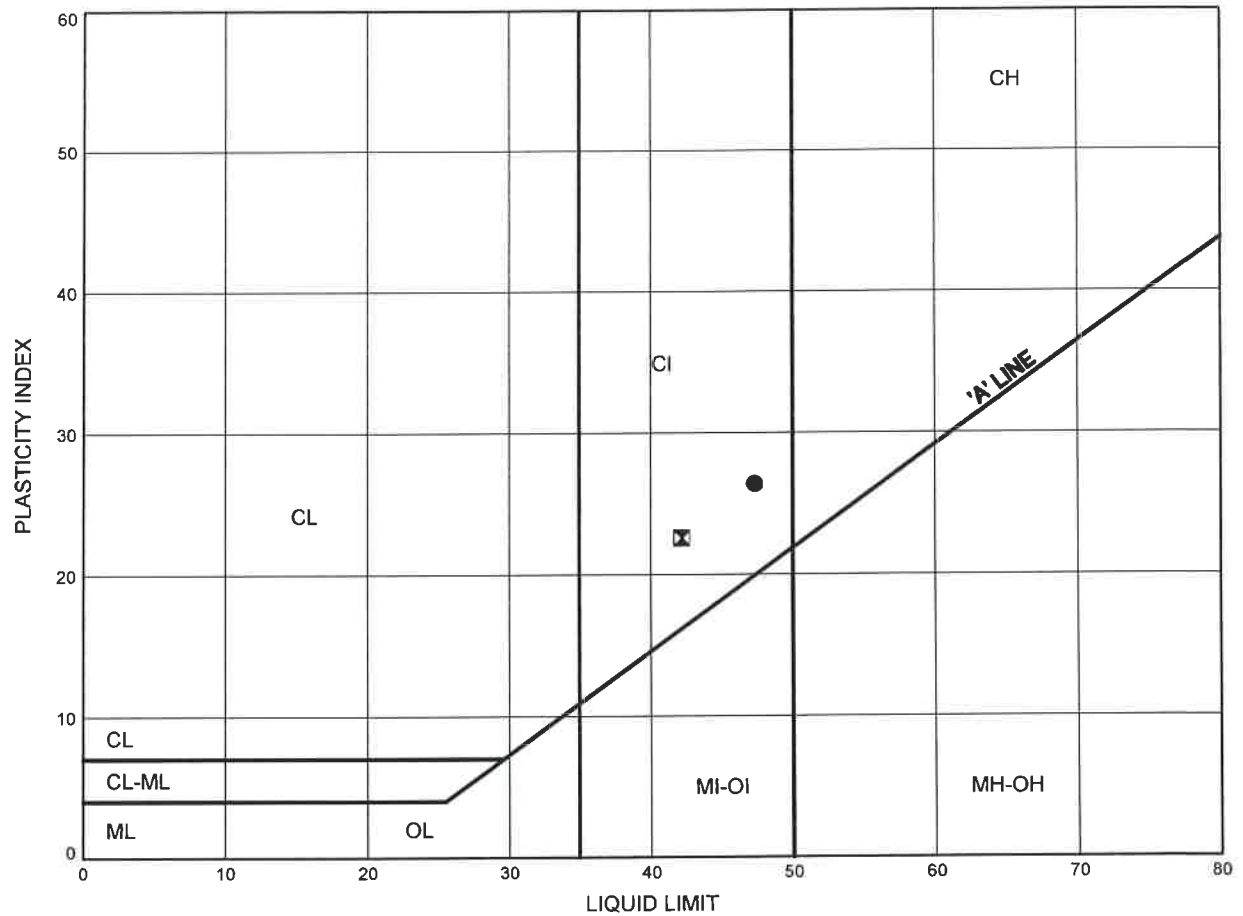




# Cloud River Culvert ATTERBERG LIMITS TEST RESULTS

FIGURE B5

## SILTY CLAY FILL

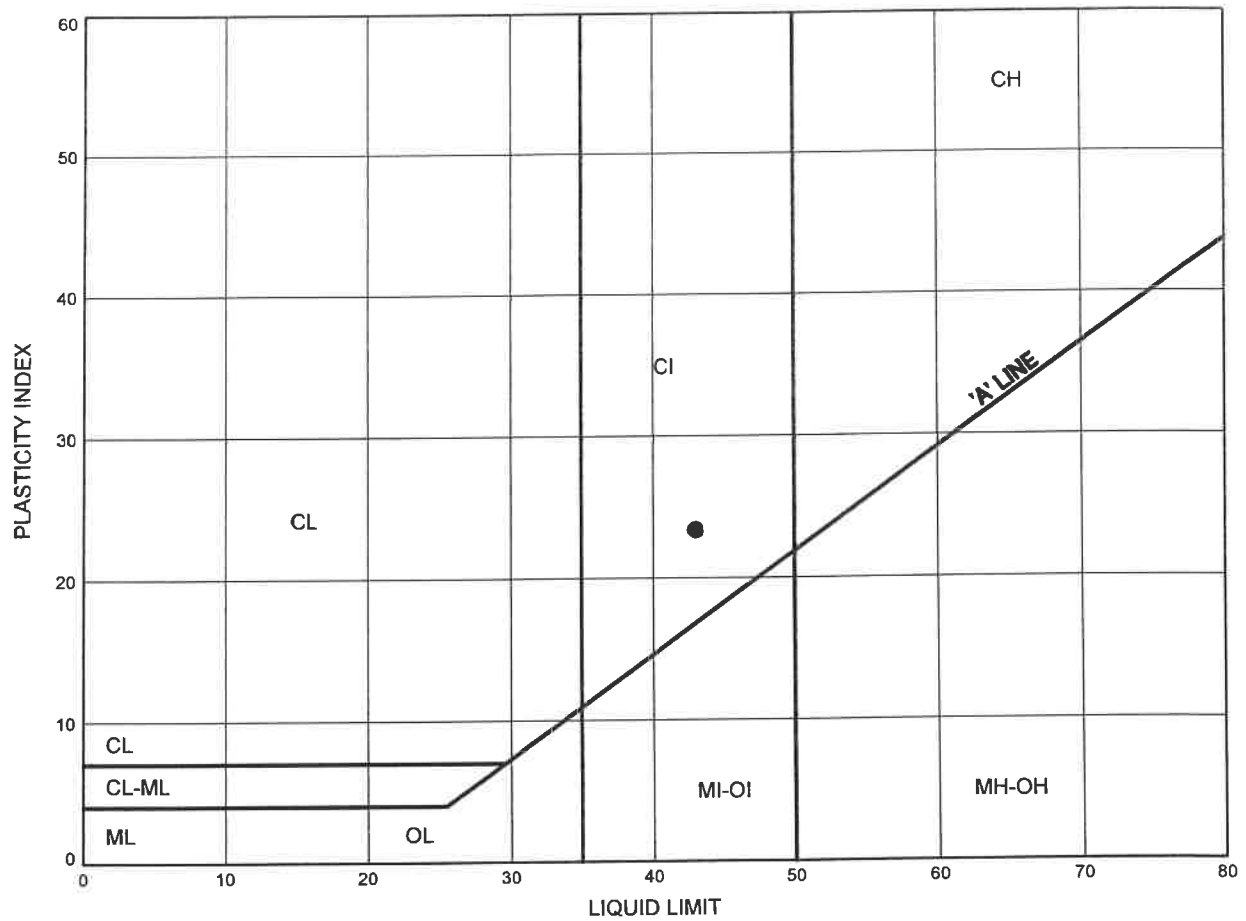


SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	CLD-02	6.40	200.40
⊠	CLD-03	2.59	204.91

Cloud River Culvert  
**ATTERBERG LIMITS TEST RESULTS**

FIGURE B6

**SILTY CLAY**



SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	CLD-03	23.16	184.34

**Appendix C**  
**Site Photographs**



**Photograph 1** – Highway 61 and Cloud River Culvert crossing



**Photograph 2** – Highway 61 west embankment at Cloud River Culvert





**Photograph 3** – West end Cloud River Culvert - Inlet





Photographs 4 and 5 – Cloud River



**Photograph 6 – East end Cloud River Culvert - Outlet**



Cloud River Culvert  
Highway 61, Site 45-276C

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**Photographs 7 and 8 – Highway 61 embankment at Cloud River Culvert**

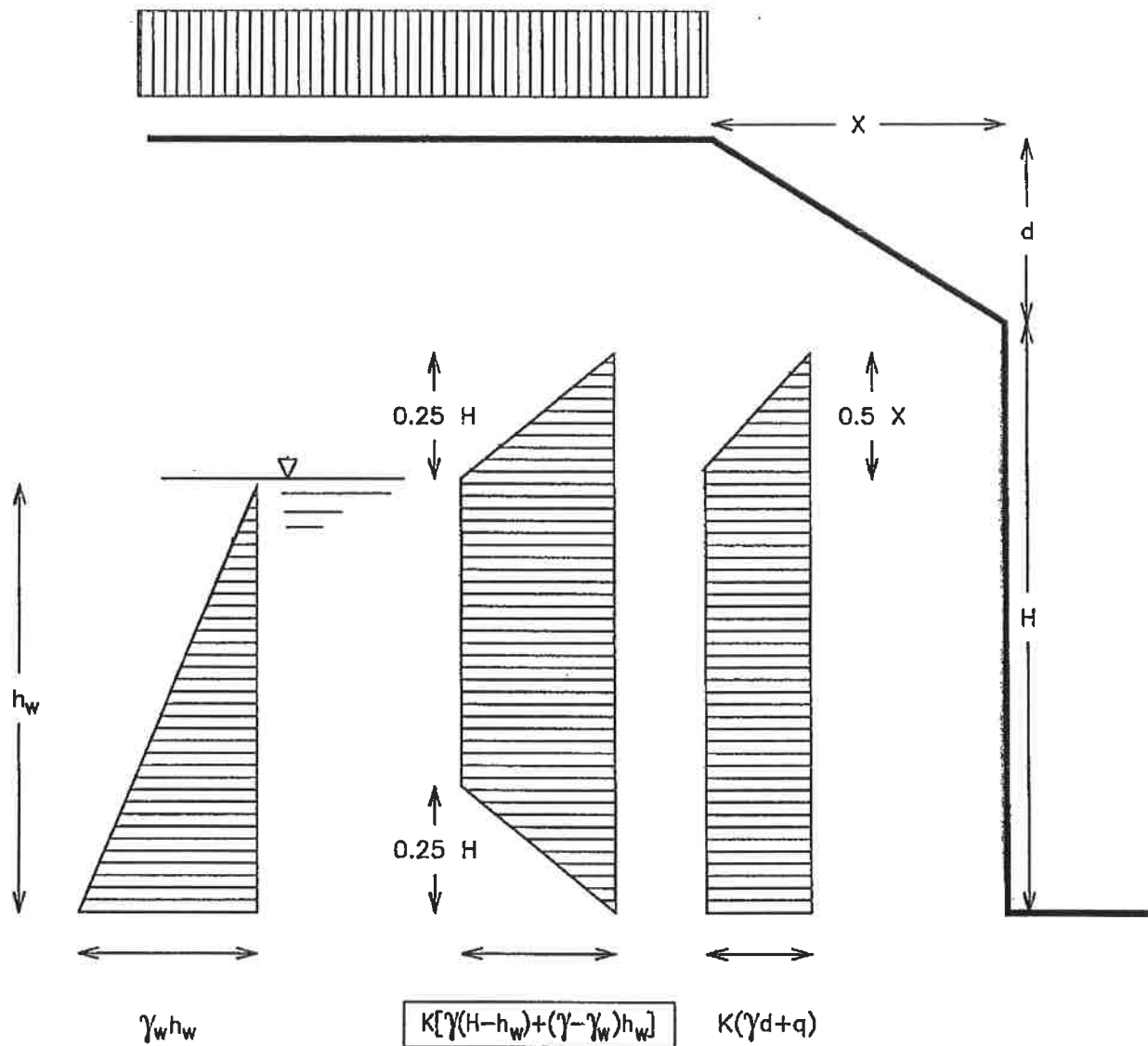
**Appendix D**  
**Foundation Comparison**

**COMPARISON OF FOUNDATION ALTERNATIVES FOR EACH FOUNDATION ELEMENT**

Preferred foundation	Open Footings Culvert Founded on		
Precast Panel on Sheet Piles	Footing on Native Soil	Augered Caissons (drilled shafts)	Driven H-Piles
<p><b>Advantages:</b></p> <ul style="list-style-type: none"> <li>i. Minimizes potential for disturbance of streambed.</li> <li>ii. Ease of construction.</li> <li>iii. Provides shoring and foundation elements in one operation.</li> <li>iv. Installation of piles could continue in freezing weather.</li> <li>v. Potentially minimizes volume of excavation and roadway protection requirements.</li> </ul> <p><b>Disadvantages:</b></p> <ul style="list-style-type: none"> <li>i. Unconventional design.</li> <li>ii. Cost of sheet piles.</li> </ul> <p><b>FEASIBLE</b></p>	<p><b>Advantages:</b></p> <ul style="list-style-type: none"> <li>i. Generally less costly construction than deep foundation elements.</li> </ul> <p><b>Disadvantages:</b></p> <ul style="list-style-type: none"> <li>i. Low available geotechnical resistance in native soils.</li> <li>ii. Deep excavation extending below the groundwater level is required.</li> <li>iii. High groundwater levels. Dewatering will be required.</li> <li>iv. Potential disturbance of river during excavation.</li> <li>v. Foundations close to river flow would be at risk due to scour and erosion</li> </ul> <p><b>NOT RECOMMENDED</b></p>	<p><b>Advantages:</b></p> <ul style="list-style-type: none"> <li>i. Construction of caissons could continue in freezing weather.</li> </ul> <p><b>Disadvantages:</b></p> <ul style="list-style-type: none"> <li>i. Suitable bearing stratum was encountered very deep (33.0 m) at this site.</li> <li>ii. Higher cost than spread footings</li> <li>iii. Specialized installation measures such as temporary liners and drilling mud will be required to install caissons in cohesionless soils under the water table.</li> <li>iv. Potential difficulty in cleaning and inspecting bases.</li> </ul> <p><b>NOT RECOMMENDED</b></p>	<p><b>Advantages:</b></p> <ul style="list-style-type: none"> <li>i. Piles will develop geotechnical resistance by shaft friction in loose to compact silt.</li> <li>ii. Installation of piles could continue in freezing weather.</li> <li>iii. Foundation construction may require less volume of excavation than footings.</li> <li>iv. Readily installed.</li> </ul> <p><b>Disadvantages:</b></p> <ul style="list-style-type: none"> <li>i. Higher unit costs than footings.</li> <li>ii. Relatively low axial and lateral resistance available.</li> </ul> <p><b>FEASIBLE</b></p>

**Appendix E**  
**Figures for Lateral Earth Pressure Distribution**

SURCHARGE  $q$



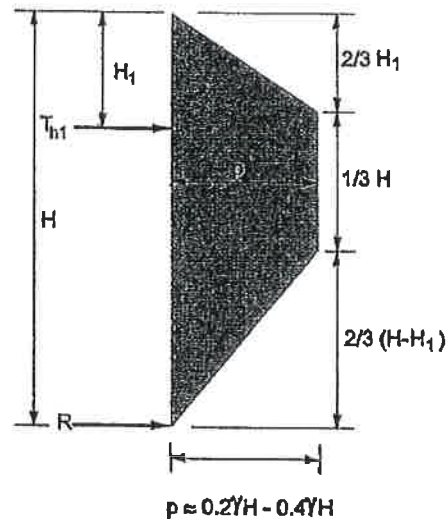
$\gamma$  = UNIT WEIGHT OF CLAY  
 $\gamma_w$  = UNIT WEIGHT OF WATER  
 $K$  = EARTH PRESSURE COEFFICIENT = 0.2 TO 0.4

# LATERAL PRESSURE DISTRIBUTION BRACED SHORING IN STIFF CLAYS

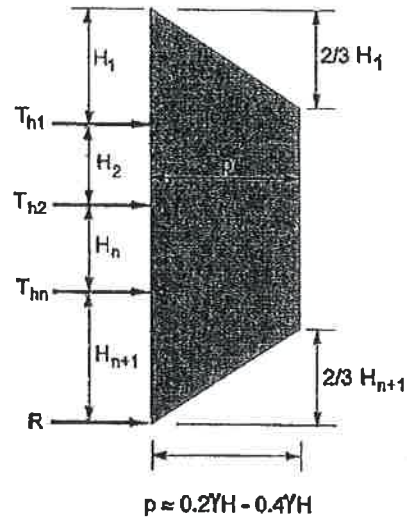


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ENGINEER:	RPR	DRAWN:	MFA	APPROVED:	-
DATE:	SEPTEMBER 2011	SCALE:	N.T.S.	DRAWING No.	FIGURE 1



(a) Walls with one level of ground anchors



(b) Walls with multiple levels of ground anchors

$H_1$  = Distance from ground surface to uppermost ground anchor

$H_{n+1}$  = Distance from base of excavation to lowermost ground anchor

$T_H$  = Horizontal load in ground anchor I

$R$  = Reaction force to be resisted by subgrade (i.e., below base of excavation)

$p$  = Maximum ordinate of diagram

TOTAL LOAD (kN/m/meter of wall) =  $3H^2 - 6H^2$  (H in meters)

. Recommended apparent earth pressure envelope for stiff to hard clays.



## APPARENT EARTH PRESSURE ENVELOPE

From FHWA-IF-99-015  
Geotechnical Eng. Circular No. 4  
Ground anchors and anchored systems

**FIGURE 2**

## **Appendix F**

### **List of SPs and OPSS, and Suggested Text for NSSP**

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

- OPSS 903, November 2009
- OPSD 803.010
- OPSS 501 dated November 2010
- OPSS 804, November 2010
- OPSS 902, November 2010
- OPSS 539

**2. Suggested Text for NSSP on sheet pile installation**

Vibratory equipment must not be used for sheet pile installation.



## **Appendix G**

### **Drawing Borehole Locations and Soil Strata**

