



THURBER ENGINEERING LTD.

**FOUNDATION INVESTIGATION AND DESIGN REPORT
REPLACEMENT OF STRUCTURAL CULVERT No 3-446C
800 m WEST OF DEVINE ROAD
HWY 417 FROM EIGHTH LINE TO LIMOGES
OTTAWA, ONTARIO
G.W.P. 4064-06-00**

GEOCRES Number: 31G-250

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To
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PART 1: FACTUAL INFORMATION

1 INTRODUCTION

This report presents the factual data obtained from a foundation investigation conducted by Thurber Engineering Ltd. (Thurber) at a culvert replacement site on Highway 417 near Ottawa, Ontario.

The foundations terms of reference indicate that there is no record of any previous foundation investigation carried out at or near the subject culvert.

A foundation investigation has been carried out by Thurber Engineering Ltd. (Thurber) for the design of the replacement of Structural Culvert #3-446C on Highway 417 approximately 800 m west of Devine Road (County Road 8), near Ottawa, Ontario. The existing culvert is located at approximate chainage 14+215 EB and 14+266 WB in Cumberland Township. Thurber carried out the investigation as a sub-consultant to URS Canada Inc. (URS) under Agreement No. 4012-E-0001, Part B.

The purpose of this investigation was to explore the subsurface conditions at the site and, based on this data, to provide a borehole location plan, record of boreholes, a stratigraphic profile, laboratory test results and a written description of the subsurface conditions.

2 SITE DESCRIPTION

The site is located within a Physiographic Region known as the Russell and Prescott Sand Plains which are characterized by a sand deposit ranging from 3 to 9 m thick underlain by deep marine clay deposits.

In the vicinity of the culvert site, Highway 417 has a rural freeway configuration with two lanes in each direction, separated by a vegetated median which is approximately 24 m wide. The existing roadway platform for both the eastbound and westbound direction includes two 3.75 m wide lanes, a

median shoulder 1.6 m wide and an outside shoulder of 2.9 m. The culvert site is located within a tangent section. The topography is flat with agricultural fields on the south side and light brush on the north side.

Drainage is currently provided through a 3.9 m span, 2.6 m high, multi-plate pipe arch culvert located 10 m to 40 m west of the proposed crossing. Flanking the pipe arch culvert are twin 1.5 m diameter CSP culverts. It is understood that the condition of these existing pipes has deteriorated to the point where replacement is warranted. The embankment side slopes near the existing culverts are vegetated and in good condition, as shown in the photographs provided in Appendix D. The roadway is in fair condition at this location with no dips or bumps present above the existing culverts. The roadway embankment is less than 3.5 m in height.

3 SITE INVESTIGATION AND FIELD TESTING

No historical geotechnical data was available from the MTO GEOCREST library for this site.

The field investigation for this site included three boreholes drilled on July 23rd and 24th, 2014 for the proposed alignment to the east of the existing culvert.

The locations of the boreholes are shown on the Borehole Location and Soil Strata drawing in Appendix A and summarized in the table below.

Location	Borehole	Depth (m)
Near Proposed Inlet	14-1a	10.1
Median ditch	14-2a	12.2
Near Proposed Outlet	14-3a	10.2

The borehole drilling was carried out using a track-mounted drill rig supplied and operated by a specialist drilling contractor. Soil drilling was carried out using hollow stem augers. Soil samples were obtained using a 50 mm outside diameter split spoon sampler advanced in accordance with the Standard Penetration Test (SPT). In-situ shear vane testing was carried out using an MTO N-vane within soft to firm cohesive deposits.

The field work was supervised on a full-time basis by a member of our field staff who located the boreholes in the field, cleared borehole locations of underground utilities, supervised the drilling, sampling and in-situ testing operations, and logged the boreholes. The soil samples were identified in the field, placed in appropriately labelled containers and transported back to Thurber's laboratory for further examination and testing.

The locations of the boreholes and ground surface elevations at the borehole locations were surveyed by Thurber.

4 LABORATORY TESTING

Geotechnical laboratory testing was carried out in the Stantec laboratory in Ottawa, Ontario, and consisted of natural moisture content determination and visual identification of all soil samples in accordance with the current MTO standards. Grain size distribution analysis and Atterberg limit testing were also carried out to MTO and ASTM standards.

It is noted that this is a culvert replacement project with no permanent grade raise proposed. Consolidation testing was therefore not warranted since induced stresses are not increased.

The laboratory test results are presented on the records of boreholes in Appendix B and the figures in Appendix C.

5 DESCRIPTION OF SUBSURFACE CONDITIONS

5.1 General

Reference is made to the Record of Borehole sheets in Appendix B for details of the soil stratigraphy encountered in the boreholes. A stratigraphic profile for the culvert replacement alignment is presented on the Borehole Locations and Soil Strata Drawing in Appendix A for illustrative purposes. An overall description of the stratigraphy is given in the following paragraphs; however, the factual data presented in the record of boreholes governs any interpretation of the site conditions.

The results indicate that the soil stratigraphy consists of a layer of fill over a deposit of clay to silty clay over a silty sand with gravel deposit. A thin layer of peat was found at the proposed inlet borehole. Two of the boreholes were terminated within the clay/ silty clay deposit. The median borehole was terminated within the underlying silty sand with gravel deposit.

5.2 Fill

The surficial materials in all boreholes consisted of a 50mm to 100mm thick layer of rootmat over a loose to compact fill deposit. The fill ranged from silty sand with gravel to silty clay with sand. The thickness of this deposit ranged from 0.9 to 1.4 m. The base of the fill deposit was encountered at elevations ranging from 73.2 to 74.3 m.

SPT N-values ranged from 7 to 21 blows per 0.3 m penetration indicating a loose to compact deposit. The water content of the samples tested ranged from 7 % to 24 %. Grain size analyses were conducted on two samples of this material and the results are presented on Figure 1 in Appendix C and summarized in the following table.

Soil Particles	%
Gravel	0 to 23
Sand	13 to 45
Silt	19 to 41
Clay	13 to 46

Two samples were tested to determine the Atterberg limits. The results are presented on Figure 5 in Appendix C and summarized in the table below. The samples tested are of low to intermediate plasticity.

Plastic Limit	13 to 20
Liquid Limit	21 to 37
Plasticity Index	8 to 17

5.3 Peat / Silty Sand

The surficial layer of fill was underlain by a thin layer of peat in the borehole located near the proposed inlet, 14-1a. The layer was 60mm thick and terminated at an elevation of 73.9 m.

In the borehole located near the proposed outlet, 14-3a, a 75mm thick layer of silty sand with trace organic material was found below the fill layer. The layer terminated at an elevation of 74.2 m.

5.4 Silty Clay

The peat/ silty sand deposits in Boreholes 14-1a and 14-3a, were underlain by a layer of brown, oxidized silty clay. The surficial fill in Borehole 14-2a, was underlain by the same layer. The thickness of this deposit ranged from 1.1 to 2.2 m. The base of the deposit was at elevations ranging from 71.7 to 72.8 m.

SPT N-values ranged from weight of hammer to 6 blows per 0.3 m penetration and two in-situ vane shear tests indicated shear strength of 17 kPa with a sensitivity of 5. This indicates a firm to soft consistency. The water content of the samples tested ranged from 24 % to 51 %. Grain size analyses were conducted on four samples of this material and the results are presented on Figure 2 in Appendix C and summarized in the following table.

Soil Particles	%
Gravel	0
Sand	1 to 5
Silt	34 to 51
Clay	46 to 65

Four samples were tested to determine the Atterberg limits. The results are presented on Figure 6 in Appendix C and summarized in the table below.

Plastic Limit	17 to 19
Liquid Limit	34 to 53
Plasticity Index	17 to 34

The composition of the deposit is generally described as silty clay with low to high plasticity.

The moisture content, Atterberg Limit and undrained shear strength data are indicative of a clay which is slightly over-consolidated.

5.5 Clay

The silty clay layer was underlain by a grey clay deposit in all of the boreholes. Boreholes 14-1a and 14-3a were terminated within the clay deposit at depths of 10.1 and 10.2 m below ground surface (elevation 64.9 to 65.2 m). The clay layer was fully penetrated in Borehole 14-2a where the thickness of the clay was 8.1 m and the base elevation was 63.6 m.

The SPT N-values were consistently weight of hammer with the exception of one sample that straddled the grey clay and the overlying crust where an N-value of 1 blows per 0.3 m penetration was recorded. The results of in-situ vane shear testing indicated undrained shear strengths ranging from 15 to 50 kPa indicating that the grey clay has soft to firm consistency, typically with shear strength increasing with depth. The sensitivity of the clay ranged from 0 to 18, however, it is noted that at some locations the strength of the clay was too low to permit remolded vane tests to be carried out. The moisture content of the grey clay ranged from 57 % to 95 %.

Four samples of this deposit were subjected to gradation analysis. The results are summarized in the table below and presented on Figure 3 in Appendix C.

Soil Particles	%
Gravel	0
Sand	1 to 2
Silt	16 to 32
Clay	66 to 83

Four samples were tested to determine the Atterberg limits. The results are presented on Figure 7 in Appendix C and summarized in the table below.

Plastic Limit	19 to 23
Liquid Limit	46 to 67
Plasticity Index	27 to 44

The sample taken from Borehole 14-3a at a depth of 9.45 m can be classified as having intermediate plasticity (CI) while all other samples can be classified as having high plasticity (CH).

The moisture content, Atterberg Limit and undrained shear strength data are indicative of a clay which is marginally over-consolidated.

5.6 Silty Sand with Gravel (SM)

The grey clay layer in Borehole 14-2a was underlain by a layer of grey silty sand with gravel (SM). The borehole was terminated within this layer at a depth of 12.2 m below ground surface (elevation of 62.4 m).

SPT N-values were 66 and 21 blows per 0.3 m penetration indicating a very dense to compact deposit. The water content of the samples tested were 8 % and 9 %. Grain size analysis was conducted on a sample of this material and the results are presented on Figure 4 in Appendix C and summarized in the following table.

Soil Particles	%
Gravel	37
Sand	40
Silt	18
Clay	5

5.7 Groundwater Conditions

Free water was observed in Borehole 14-3a at a depth of 2.6 m below ground surface (elevation of 72.6 m) at the time of drilling.

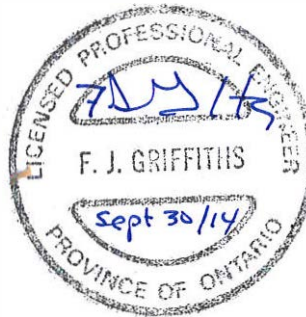
The values are short-term readings and seasonal fluctuations of the groundwater level are to be expected. In particular, the groundwater level will be influenced by the water level in the stream and ditches and may be at a higher elevation after the spring snowmelt or after periods of heavy rainfall.

The general arrangement drawing indicates a water level of 72.64 m and a high water level of 74.13 m in the replacement culvert.

6 MISCELLANEOUS

Thurber staked and/or marked the borehole locations in the field and obtained utility clearances prior to drilling. Thurber surveyed the borehole locations, and provided the northing and easting coordinates and ground surface elevations. George Downing Estate Drilling Ltd. of Hawkesbury, Ontario supplied and operated a track-mounted CME 75 drill rig to carry out the drilling, sampling, and in-situ testing. The drilling, and sampling operations in the field were supervised on a full time basis by Mr. Nick Weil of Thurber. Laboratory testing was carried out by Stantec in its MTO-approved laboratory in Ottawa.

Overall project management and direction of the field program was provided by Dr. Fred Griffiths, P.Eng. Interpretation of the field data and preparation of this report was completed by Dr. Fred Griffiths, 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 GENERAL

This report presents interpretation of the geotechnical data in the factual report and presents foundation recommendations for the design of the replacement of Culvert #3-446C on Highway 417 approximately 800 m west of Devine Road (County Road 8), near Ottawa, Ontario. Site photographs are presented in Appendix D.

Historical contract drawings (Contract No. 69-209, W.P. No. 34-66-03) indicate the following details regarding the existing structure:

- It consists of three culverts which cross the highway at a skew (approximately 30 degrees from perpendicular). The three culverts are continuous beneath the eastbound lanes, westbound lanes and median with a total length of 242 feet (73.8 m). The three culverts consist of a 12'10" x 8'7" structural pipe arch culvert in the middle with 60" diameter CSP culverts located on both sides. The invert elevations are as follows:

Culvert	Invert Elevations	
	South Side	North side
Multi-plate pipe arch	237.70 ft / 72.45 m	237.00 ft / 72.24 m
60" CSP culverts LT & RT	241.30 ft / 73.55 m	240.60 ft / 73.33 m

- Ditch inlets are present within the median ditch on both the east and west sides and outlet into the CSP culverts.
- All three culverts were designed to be installed within a single benched excavation that was proposed to be backfilled entirely with granular material, including 10:1 frost tapers. The

design cover appears to be less than 3 feet. The bedding material was proposed to be placed to approximately 0.46 m below the invert of the arch culvert.

- The culvert end treatment is indicated as consisting of cast-in-place concrete headwalls at both ends, however, concrete headwalls are not currently present (see photos in Appendix D).

Based on the General Arrangement Drawing dated September 2014 provided by URS the three existing steel culverts will be replaced on a new, skewed alignment to the east by constructing twin 3000 x 2400 mm concrete culverts extending continuously beneath the east bound lanes (EBL), west bound lanes (WBL) and centre median, for a total length of 105 m. Water will flow through the culvert from south to north with the proposed stream bed elevations at 72.34 m at the inlet and 72.13 m at the outlet. The stream bed elevations include consideration of the 300 mm of stream bed material placed inside the culvert for environmental reasons, thus the underside of the box culverts will be at 71.79 m at the inlet and 71.58 at the outlet. The top of pavement at centreline in the EBL and WBL is approximately elevation 75.75 m and 75.71 m respectively. There is approximately 1.2 of cover at centreline for the proposed culverts. The edge of shoulder is approximately 3.4 m above the stream bed.

The proposed construction methodology includes a pre-construction stage followed by three stages with the culverts being installed from outlet to inlet. Water flow will be maintained through the existing culverts while installing the new twin box culverts. Roadway Protection Systems running roughly parallel to the highway centrelines lanes are proposed to allow four lanes of traffic to be maintained throughout construction. Both of the existing embankments will be widened into the median as much as 7.5 m for temporary use as a detour; as much as 0.82 m of fill will be required

The preliminary general arrangement drawing provided by URS indicates a high water level at elevation 74.13 m and observed water level at elevation 72.64 m in the replacement culvert.

The discussions and recommendations presented in this report are based on the information provided by URS and on the factual data obtained during the course of this investigation.

8 SEISMIC CONSIDERATIONS

The following seismic parameters should be used for design:

- | | |
|-------------------------------------|------|
| • Velocity Related Seismic Zone | 2 |
| • Zonal Velocity Ratio | 0.10 |
| • Acceleration Related Seismic Zone | 4 |
| • Zonal Acceleration Ratio | 0.20 |
| • Peak Horizontal Acceleration | 0.2g |

The Soil Profile Type at this site has been classified as Type III based on the presence of 9 to 12 m of soft to medium clay.

The potential for liquefaction of the silty clay deposit has been assessed using the criteria presented by Bray et al. (2004). Based on these criteria, the deposit is considered not susceptible to liquefaction due to its plasticity.

9 FOUNDATIONS

Based on the borehole records and proposed invert elevations, the base of the culvert structure will be within the soft grey silty clay deposit.

9.1 Culvert Foundation Alternatives

This section presents discussions on alternate types of culvert foundations, and provides recommendations on feasible and/or preferred foundation options.

Culvert Rehabilitation with Slip Lining would result in some loss of pipe diameter and thus reduce the overall capacity of the culverts. It is understood that this is not hydraulically feasible at this site. Pipe bursting is also considered to be unfeasible as the existing main pipe has an arched section as opposed to circular and is constructed of steel. Furthermore there is very little soil cover over the existing pipe, thus heave is a concern.

A comparison of the typical alternatives, based on their respective advantages and disadvantages, is included in Appendix E.

Concrete, Open Footing Culvert

Concrete, open footing culverts founded directly on the soft clay are not considered feasible at this site due to the low bearing resistance available and potential for large settlements.

Concrete, Open Footing Culvert on Granular Pads

Concrete, open footing culverts founded on granular pads are feasible at this site but would require deeper and wider excavations, thereby increasing the cost of protection and dewatering systems.

Concrete, Open Footing Culvert on Deep Foundations

An open bottom culvert could be supported on deep foundations such as driven steel piles extending to refusal on the underlying dense layer. However, this foundation alternative would be far more expensive than a concrete closed box culvert.

Concrete Box (Closed) Culvert

Concrete box (closed) culverts are the recommended alternative at this site from both a cost and technical perspective.

Given the large traffic volumes, there are construction timing advantages of precast boxes in comparison to cast-in-place concrete construction, thus it is recommended that precast box culverts be utilized for this project.

Foundation recommendations for pre-cast concrete closed box culverts are provided in the following section. Construction of pre-cast concrete box culverts should be carried out in accordance with OPSS 422.

9.2 Box Culvert on Granular Pad

The anticipated founding level for the pre-cast box culvert sections ranges from 71.79 m at the inlet to 71.58 m at the outlet. This elevation is within the soft to firm clay deposit, thus a thicker than typical working pad is recommended. Allowing for 150 mm of Granular A over 600 mm of Granular B Type II bedding, the base of excavation is expected to be at approximately elevation 71.04 to 70.83 m. Foundation bearing resistance recommendations for the culvert are provided below:

Table 9.1 – Foundation Bearing Resistance Recommendations- Box Culvert

Box Culvert Width (m)	Factored Geotechnical Resistance at ULS (kPa)	Bearing Pressure at SLS (kPa)
3.5 + 3.5	110	75

The factored geotechnical resistance at ultimate limits states (ULS) includes a resistance factor of 0.5.

The bearing pressure at serviceability limit states (SLS) corresponds to a total settlement of 25 mm.

The geotechnical resistances are based on a footing subjected to vertical concentric loading. Where eccentric or inclined loads are applied, the resistance used in the design must be reduced in accordance with the CHBDC Clauses 6.7.3 and 6.7.4.

The placement of the granular pad and the box culvert will require dewatering since the works must be completed in the dry.

9.3 Wing Walls

The overall length of the culvert could be shortened by adding Wing Walls to the culvert at the inlet and outlet. We have examined walls extending to approximately 2.0 m above stream bed. A comparison of typical alternatives it included in Appendix E. Further discussion for the most viable alternative is presented below.

Gabion Walls:

Walls consisting of gabion baskets would be placed on granular pads. The anticipated founding level for the gabions ranges from approximately 72.0 m at the inlet to 71.8 m at the outlet. Allowing for 1.5 m of granular bedding, the base of excavation is expected to be at approximately elevation 70.5 to 70.3 m. This elevation is within the soft to firm clay deposit. Foundation bearing resistance recommendations for the culvert are provided below:

Table 9.2 – Foundation Bearing Resistance Recommendations- Gabions

Gabion Base Width (m)	Factored Geotechnical Resistance at ULS (kPa)	Bearing Pressure at SLS (kPa)
2.5	85	50

The factored geotechnical resistance at ultimate limits states (ULS) includes a resistance factor of 0.5.

The bearing pressure at serviceability limit states (SLS) corresponds to a total settlement of 25 mm.

The geotechnical resistances are based on a footing subjected to vertical concentric loading. Where eccentric or inclined loads are applied, the resistance used in the design must be reduced in accordance with the CHBDC Clauses 6.7.3 and 6.7.4.

The placement of the granular pad and the gabions will require dewatering since the works must be completed in the dry.

A slope stability analysis was carried out assuming the roadway configuration anticipated for Detour Stage 2 when traffic will be shifted slightly to the outside. The safety factor for global stability of the 2 m high gabion wall was determined to be slightly greater than 1.5 for the static case using Slide. Under seismic loading the Factor of Safety dropped to 1.0 while the under simultaneous application of seismic and detour traffic loading a Factor of Safety of 0.9 was noted.

If gabion walls are chosen the wall must also be designed for an adequate factor of safety against sliding failure. The sliding resistance between the base of the gabion basket and the granular pad should be evaluated using an unfactored coefficient of friction of 0.6. The sliding resistance between the base of the granular pad and the underlying clay should be evaluated using an unfactored coefficient of friction of 0.3.

The advantages of the gabion walls are economics and structural flexibility as gabions are able to accommodate more settlement than rigid structures. The disadvantages include very low geotechnical resistance available at ULS and SLS, marginal stability under seismic loading and construction challenges due to possible excavation basal heave, construction dewatering and a possible conflict during excavation with the existing culvert to the west on the south side.

Although, the wing walls may provide an economic benefit by shortening the culvert this would likely be offset by increased risk of construction difficulties as outlined above. It is recommended that wing walls not be included in the design.

10 BACKFILL AND LATERAL EARTH PRESSURES

Construction of the culverts should be carried out in accordance with OPSS 422.

Backfill and cover for the culverts must satisfy structural requirements but as a minimum should be carried out in accordance with OPSD 803.010. The depth of frost penetration is 1.8 m at this site.

Culvert backfill should consist of free-draining granular material conforming to OPSS Granular A, Granular B Type II or Granular B Type III specifications.

Backfill should be placed and compacted in simultaneous, equal lifts on both side of the culvert. Heavy compaction equipment should not be used adjacent to the walls and roof of the culvert. Compaction should be carried out in accordance with OPSS 501.

Static Earth Pressure

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 10.1)

γ = bulk unit weight of retained soil (see Table 10.1)

- 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. Recommended unfactored values are shown in Table 10.1. As the design is based on a closed box culvert the walls will be braced at top and bottom and the at-rest coefficient should be used to assess the lateral earth pressures.

Table 10.1 – Geotechnical design parameters for static lateral earth pressures

Parameter	Existing Embankment Fill	Granular B Types I and III	Granular A and Granular B Type II
<i>Soil Parameters</i>			
Soil Unit weight (kN/m ³)	20.0	21.2	22.8
Angle of Internal friction, Ø	30°	32°	35°
<i>Walls with Horizontal Backfill</i>			
Coefficient of earth pressure at-rest, K ₀	0.50	0.47	0.43
Coefficient of active earth pressure, K _a	0.33	0.31	0.27
Coefficient of passive earth pressure, K _p	3.0	3.3	3.7
<i>Walls with 2H:1V Backfill</i>			
Coefficient of active earth pressure, K _a	0.54	0.47	0.39

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 III 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.

Combined Static and Seismic Lateral Earth Pressure Parameters

The following recommendations are per Section 4.6.4. of the CHBDC which states that seismically induced lateral soil pressures may be calculated using the Mononobe-Okabe Method with $k_h = A/2$ for structures that allow lateral yielding and $k_h = 3/2A$ for non-yielding walls, where A is the zonal acceleration ratio. An outward displacement of $250A$ (i.e. 50 mm) would be required for the wall to be considered a yielding structure.

The following unfactored seismic earth pressure coefficients (combined static and seismic) for use in the design are provided in Table 10.2 below.

Table 10.2: Lateral Earth Pressure (Under Seismic Loads, with horizontal back slope)

Parameter	Existing Embankment Fill	Granular B Types I and III	Granular A and Granular B Type II
Soil Unit Weight, kN/m^3	20.0	21.2	22.8
Angle of Internal Friction, ϕ	30°	32°	35°
Yielding Wall			
K_{AE}	0.40	0.37	0.33
K_{AE} Load application height from base as a ratio of wall height	0.36	0.36	0.37
Non-Yielding Wall			
K_{AE}	0.66	0.62	0.55
K_{AE} Load application height from base as a ratio of wall height	0.43	0.43	0.44

The total pressure due to combined static and seismic loads acting at a specific depth below the top of the wall, d , may be determined using the following equation and including consideration of material properties and the soil profile:

$$\sigma_h = K_a \gamma d + (K_{AE} - K_a) \gamma (H - d)$$

where:

σ_h = Lateral earth pressure at depth, d , below the top of the wall (kPa)

K_a = Static active earth pressure coefficient

K_{AE} = Combined static and seismic earth pressure coefficient

γ = Unit weight of the backfill soil (kN/m^3)

H = Total height of the wall (m)

d = Depth below the top of the wall (m)

The factors in the table above are “ultimate” values and require certain movements for the respective conditions to be mobilized. The values to use in design can be estimated from Figure C6.9.1 (a) in the Commentary to the Canadian Highway Bridge Design Code.

11 EMBANKMENT DESIGN AND CONSTRUCTION

Slope stability analyses have been carried out for a 3.4 m high embankment sloped at 2H:1V. Factors of Safety of 1.1 and greater than 1.5 were generated for the seismic and static cases respectively. Both cases assumed truck loading with the roadway configuration of Detour Stage 2

Embankment reconstruction, after culvert installation, should be carried out in accordance with OPSS 206. The embankment material should consist of imported Granular A or B Type II material. Excavated granular fill may also be reused as backfill provided it is not contaminated with organic or cohesive soils and there is sufficient space to stockpile on site and control the moisture content within acceptable limits for compaction.

11.1 Subgrade Preparation

Based on the proposed invert elevations, the base of excavation is expected to be within soft sensitive clay that is easily disturbed by construction activities. After removal of the existing fill and excavation to the design founding elevation (underside of bedding), the exposed surface must be inspected to confirm that the subgrade is suitable and uniformly competent.

As noted above, the subgrade is expected to consist of soft sensitive clay. Construction equipment should not be permitted to travel on the subgrade. In addition, compaction of granular bedding directly above the clay subgrade is likely to result in disturbance of the clay, pumping of fines into the granular bedding and difficulty achieving the specified degree of compaction. Options for protection of the subgrade include:

- A) Placement of a Class II non-woven geotextile over the full extent of the subgrade as a separation layer prior to placement of 150 mm of OPSS Granular A over 600 mm of Granular B Type II bedding. The bedding should be levelled and tamped but not compacted; or
- B) Placement of a concrete mud slab having a minimum thickness of 75 mm over the full extent of the subgrade prior to placement of 200 mm of OPSS Granular A bedding. The bedding should be levelled and tamped but not compacted.

Option A is recommended as it presents lower risk for disturbance of the clay subgrade and less potential for movement of the culvert sections.

Culvert construction and subgrade preparation must be carried out in the dry. This work should be carried out in accordance with OPSS 902.

11.2 Settlements

It is understood that the culverts will be replaced along a new alignment with no permanent grade raise to the roadway. The hydraulic opening represents a decrease in load beneath the new culvert thus no significant settlement is anticipated provided the subgrade is protected from disturbance during construction.

The existing culvert is to be decommissioned by removal and backfilling the excavation. The fill utilized for this purpose should consist of OPSS Select Subgrade Materials to 1.8 m from finished grade. OPSS Granular B Type I should be utilized to the underside of the pavement subbase layer. After backfilling the underlying soils will settle in response to the additional load. As much as 25 mm of settlement may occur during the first year after fill placement. Asphalt padding may be required to correct the dip that will form at this location.

The General Arrangement drawing indicates that detours are to be constructed into the median to facilitate replacement of the culvert. It is estimated that the proposed edge of pavement of the detour will be approximately 7.5 m from the existing centre line. We have examined the grades shown on the staging typicals and assumed that the crossfall of the detour will be no flatter than 2%. The existing embankment side slopes are quite flat into the median thus the increase in grade to create the detour is estimated to be approximately 0.8 m. There are soft and compressible soils at this site and settlement should be anticipated if the detour fills were to remain in place in the long term. In the short term, however, it is expected that the settlement induced by the roadway widening will be tolerable (less than 25 mm). The detour material should be removed upon completion of the culvert installation.

12 EROSION CONTROL

Erosion protection should be provided at the culvert inlet and outlet areas. Design of the erosion protection measures must consider hydrologic and hydraulic factors and should be carried out by specialists experienced in this field.

Typically, rip-rap should be provided over all surfaces with which creek water is likely to be in contact. Treatment at the outlets should be in accordance with OPSD 810.010. A vegetation cover should be established on all other exposed earth surfaces to protect against surficial erosion in general accordance with OPSS 804.

It is recommended that a clay seal or a concrete cut-off wall be used to minimize the potential for erosion near the inlet area. The clay seal should extend from the base of the bedding layer to a minimum of 0.3 m above the high water level and laterally for the width of the granular material, and have a minimum thickness of 0.5 m. The material requirements should be in accordance with OPSS 1205. A geosynthetic clay liner may be used as a clay seal.

13 EXCAVATION AND GROUNDWATER CONTROL

13.1 General

All excavations must be carried out in accordance with the Occupational Health and Safety Act (OHSA). The existing embankment fill and upper stiff clay is considered Type 3 soil as per the OHSA. For the purposes of the OHSA, the deeper soft sensitive clay is considered Type 4 soil. As the bottom of the excavations will extend into the soft grey clay they will require excavation side slopes at 3H:1V where open cut techniques are proposed.

13.2 Excavations

Excavations for culvert replacement will typically be carried out through the existing embankment fill and extend into the underlying native soils. Protection systems will be required to facilitate the proposed construction staging. Protection systems should be designed by a licensed Professional Engineer experienced in such designs. OPSS 539 “Construction Specifications for Protection Systems” must be referenced in the contract documents. It is recommended that Performance Level 2, as per Clause 539.04.01.01 (maximum horizontal displacement of 25 mm), be specified for this culvert replacement site.

It is noted that there is a risk of basal instability from the soft to firm clay during excavation. The design depth of the shoring should be carefully selected to minimize the risk of basal heave and to provide sufficient lateral resistance.

13.3 Groundwater Control

It is expected that groundwater and surface water will accumulate in the excavations during culvert construction. The groundwater level is expected to be largely governed by the water level in the creek and seasonal weather patterns. The Contractor must make provisions to control any groundwater seepage, surface runoff and ponding by measures including the use of sump pumps, cofferdams, and creek diversion and protection systems to maintain dry excavations during the course of construction. Selection of the equipment and methodology to excavate and prepare the founding surface is the responsibility of the Contractor. The preparation of the culvert and wing wall subgrade must be carried out in the dry.

14 CONSTRUCTION CONCERNS

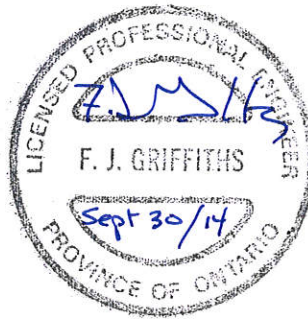
The planned construction methodology includes staged construction with protection systems in order to maintain both traffic flow and water flow through the culvert area. Potential construction concerns include, but are not necessarily limited to, the following:

- Impact of excavation on the existing pavement surface. Daily visual inspection of the pavement surface must be carried out in the vicinity of the culvert construction. If cracks form in the pavement or settlement is observed to occur, these matters must immediately be brought to the attention of the C.A. for determining the level of remedial action that is required.
- Implementation of an adequate and effective surface water management and dewatering plan to construct the replacement culverts and subgrade in the dry.
- Removal of organics, soft soils and alluvial deposits near creek/stream channels.
- The base of excavations for the culverts will be within soft clay soils that are easily disturbed.
- Confirmation that the culvert backfill is adequately placed and compacted to specifications.

The successful performance of the culvert will depend largely upon good workmanship and quality control during construction. Observation of the excavation and backfilling operations by the QVE will be required during construction to confirm that the foundation recommendations are correctly implemented and material specifications are met.

15 CLOSURE

Preparation of this foundation design report was carried out by Fred Griffiths, P.Eng. The report was reviewed by Dr. P.K. Chatterji, P.Eng.



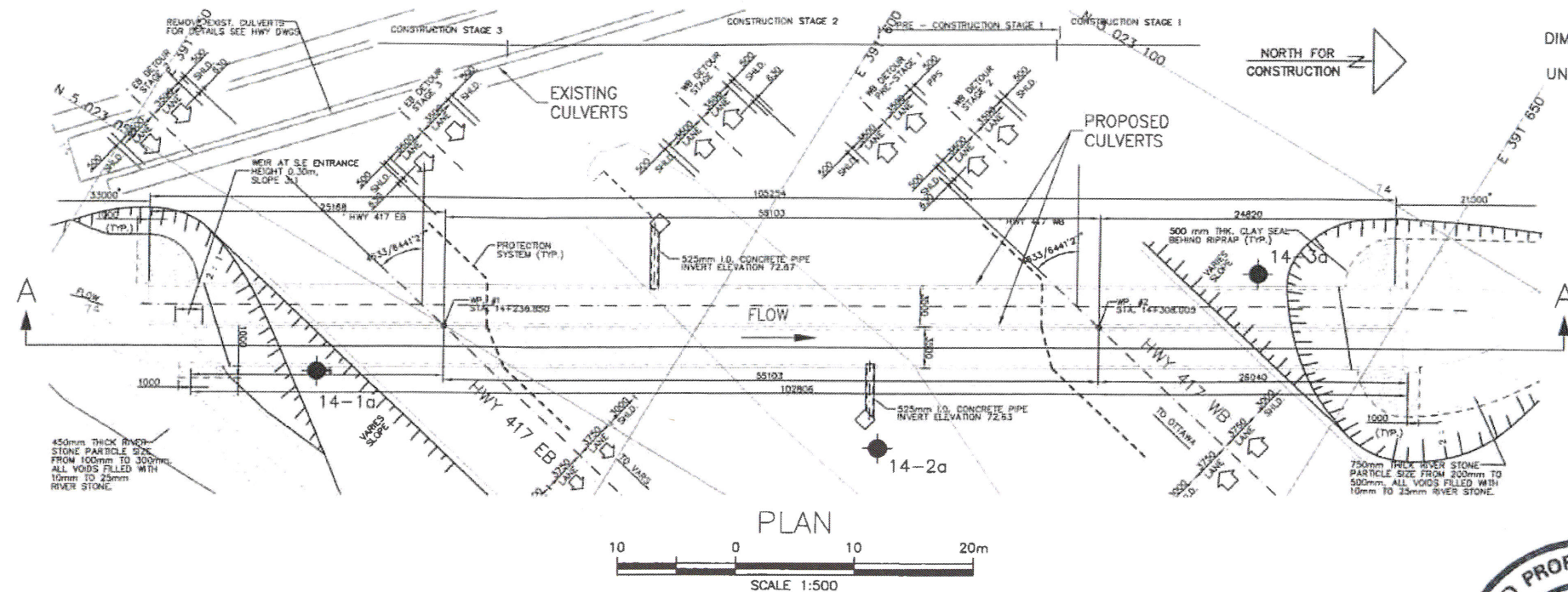
Fred J. Griffiths, P.Eng. Ph.D.
Associate / Senior Geotechnical Engineer



P.K. Chatterji, P.Eng. Ph.D.
Principal, Designated MTO Contact

Appendix A
Borehole Locations and Soil Strata Drawings

19-4406-6



METRIC

DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN

NORTH FOR
CONSTRUCTION

CONT No
GWP No 4012-E-0001

HIGHWAY 417
SHAW CREEK CULVERT
REPLACEMENT
BOREHOLE LOCATIONS AND SOIL STRATA

SHEET
265

URS

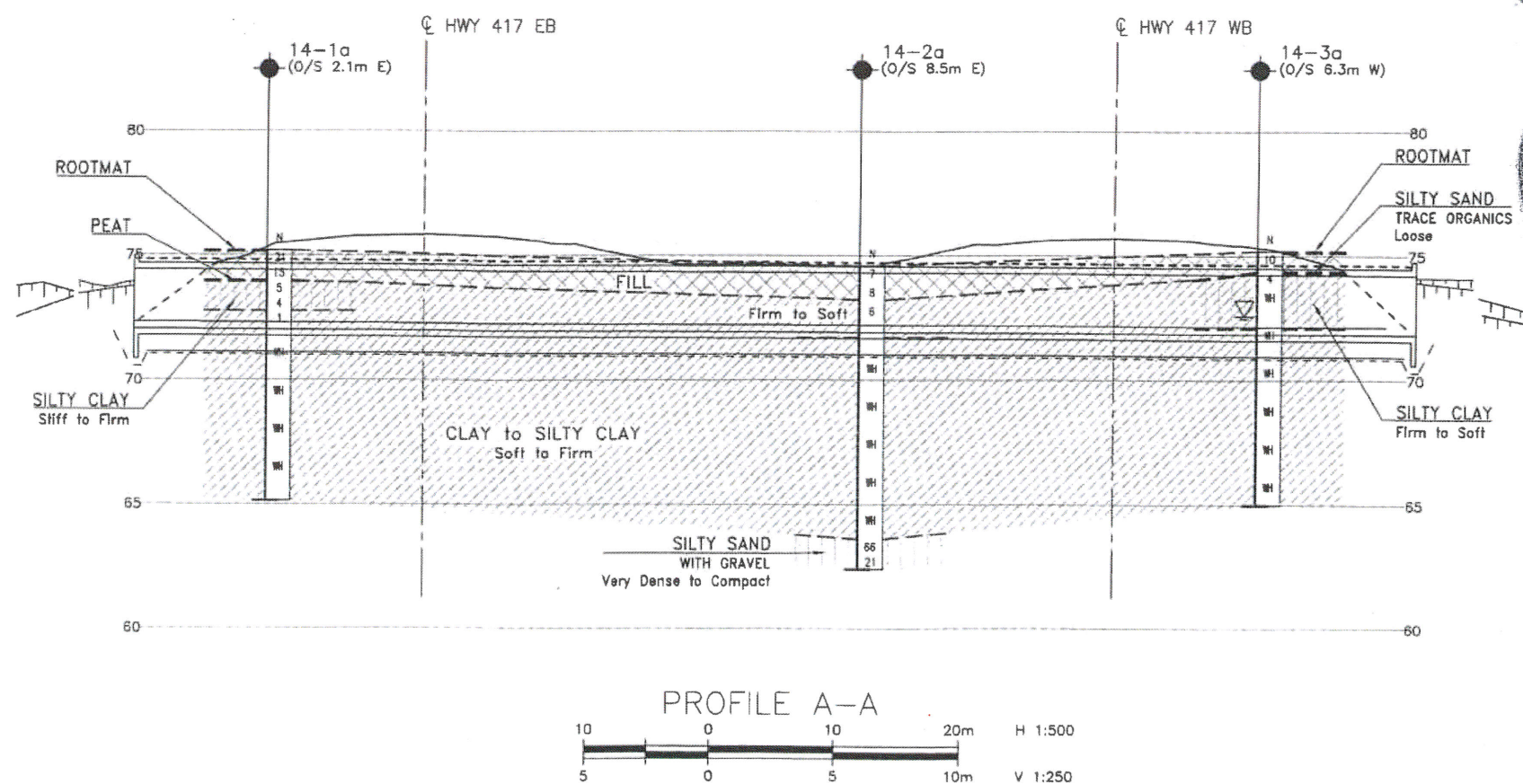


THURBER ENGINEERING LTD



KEYPLAN

LEGEND








70514

F. J. GRIFFITHS

Sept 26/14

PROVINCE OF ONTARIO

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

[illegible]

-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. 31G-250

REVISED								
	DATE	BY	DESCRIPTION					
	DESIGN	GM	CHK	PC	LOAD			DATE SEP 20
	DRAWN	MFA	CHK	CM	SST	INSTRUC	TWINS	L

[illegible]

Appendix B
Record of Borehole Sheets

19-4406-6

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 ⁽¹⁾ '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
 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	Peat and other highly organic soils.
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			
Bedding	Bedding Plane Spacing	Rock Strength	Approximate Uniaxial Compressive Strength		Field Estimation of Hardness*
			(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.

TERMS					
Total Core Recovery: (TCR)	Core recovered as a percentage of total core run length.	Weak	5.0 to 25.0	750 to 3,500	Can be peeled by a pocket knife with difficulty
Solid Core Recovery: (SCR)	Percent Ratio of solid core of full cylindrical shape recovered. Expressed with respect to the total length of core run.	Very Weak	1.0 to 5.0	150 to 750	Can be peeled by a pocket knife, crumbles under firm blows of geological pick.
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.	Extremely Weak (Rock)	0.25 to 1.0	35 to 150	Indented by thumbnail
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 14-1a

1 OF 2

METRIC

GWP# 4012-E-0001 LOCATION N 5 023 041.2 E 391 574.4 ORIGINATED BY NW
 HWY 417 BOREHOLE TYPE Hollow Stem Augers COMPILED BY GM
 DATUM Geodetic DATE 2014.07.23 - 2014.07.23 CHECKED BY FG

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						
75.2								20	40	60	80	100		
0.0								○ UNCONFINED + FIELD VANE						
0.1	50mm ROOTMAT							● QUICK TRIAXIAL × LAB VANE						
	Silty SAND with gravel to Silty CLAY with sand (CI) Trace organics Brown Compact FILL		1	SS	21		75							23 45 19 13
			2	SS	15									0 13 41 46
74.0							74							
73.8	PEAT		3	SS	5									
1.3	Silty CLAY (CI) Brown Firm		4	SS	4		73							0 3 51 46
72.8			5	SS	1									
2.4	CLAY (CH) Grey Soft to Firm						72	5.0 +						
								8.0 +						
			6	SS	WH		71							
								+						
							70	10.0 +						
			7	SS	WH									0 1 16 83
							69							
								+						
								6.0 +						
			8	SS	WH		68							
								+						
							67	5.0 +						
			9	SS	WH									
							66							
								8.0 +						

Continued Next Page

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

RECORD OF BOREHOLE No 14-1a

2 OF 2

METRIC

GWP# 4012-E-0001 LOCATION N 5 023 041.2 E 391 574.4 ORIGINATED BY NW
 HWY 417 BOREHOLE TYPE Hollow Stem Augers COMPILED BY GM
 DATUM Geodetic DATE 2014.07.23 - 2014.07.23 CHECKED BY FG

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa	WATER CONTENT (%)					
65.2	Continued From Previous Page	///												
10.1	End of Borehole													

METRIC

[illegible]




+³, ×³: Numbers refer to Sensitivity

RECORD OF BOREHOLE No 14-2a

2 OF 2

METRIC

GWP# 4012-E-0001 LOCATION N 5 023 060.0 E 391 618.4 ORIGINATED BY NW
 HWY 417 BOREHOLE TYPE Hollow Stem Augers COMPILED BY GM
 DATUM Geodetic DATE 2014.07.24 - 2014.07.24 CHECKED BY FG

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			SHEAR STRENGTH kPa													WATER CONTENT (%)		
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE													W P W W L		
	Continued From Previous Page							20	40	60	80	100											
63.6	CLAY (CH) Firm to Soft Grey		9	SS	WH		64																
11.0	Silty SAND with gravel (SM) Very Dense to Compact Grey		10	SS	66		63								○								
62.4			11	SS	21										○				37 40 18 5				
12.2	End of Borehole																						

RECORD OF BOREHOLE No 14-3a

1 OF 2

METRIC

GWP# 4012-E-0001 LOCATION N 5 023 089.0 E 391 638.2 ORIGINATED BY NW
 HWY 417 BOREHOLE TYPE Hollow Stem Augers COMPILED BY GM
 DATUM Geodetic DATE 2014.07.23 - 2014.07.23 CHECKED BY FG

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 (%)				GR SA SI CL							
								○ UNCONFINED + FIELD VANE	w _P w w _L												
						● QUICK TRIAXIAL × LAB VANE															
75.2								20	40	60	80	100									
0.0																					
0.1	75mm ROOTMAT																				
0.2	SAND with gravel and rootlets Brown Compact (FILL)		1	SS	10		75														
74.3																					
74.9	Silty CLAY Brown Stiff (FILL)		2	SS	4		74										0	5	34	61	
0.9																					
	Silty SAND Trace Organics Brown Loose		3	SS	WH													0	3	44	53
	Silty CLAY (CL) Brown Firm to Soft Moist						73														
								5.0													
								5.0													
72.0							72														
3.1	CLAY (CH) to Silty CLAY (CI) Grey Soft		4	SS	WH																
							71														
								8.0													
			5	SS	WH													0	1	20	79
							70														
							69														
			6	SS	WH																
							68														
							67														
			7	SS	WH																
							66														
								18.0													
			8	SS	WH													0	2	32	66

Continued Next Page

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

RECORD OF BOREHOLE No 14-3a

2 OF 2

METRIC

GWP# 4012-E-0001 LOCATION N 5 023 089.0 E 391 638.2 ORIGINATED BY NW
 HWY 417 BOREHOLE TYPE Hollow Stem Augers COMPILED BY GM
 DATUM Geodetic DATE 2014.07.23 - 2014.07.23 CHECKED BY FG

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)				
							20 40 60 80 100 ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE					W _p W W _L 20 40 60					
64.9	Continued From Previous Page						65	8.0									
10.2	End of Borehole																

Appendix C

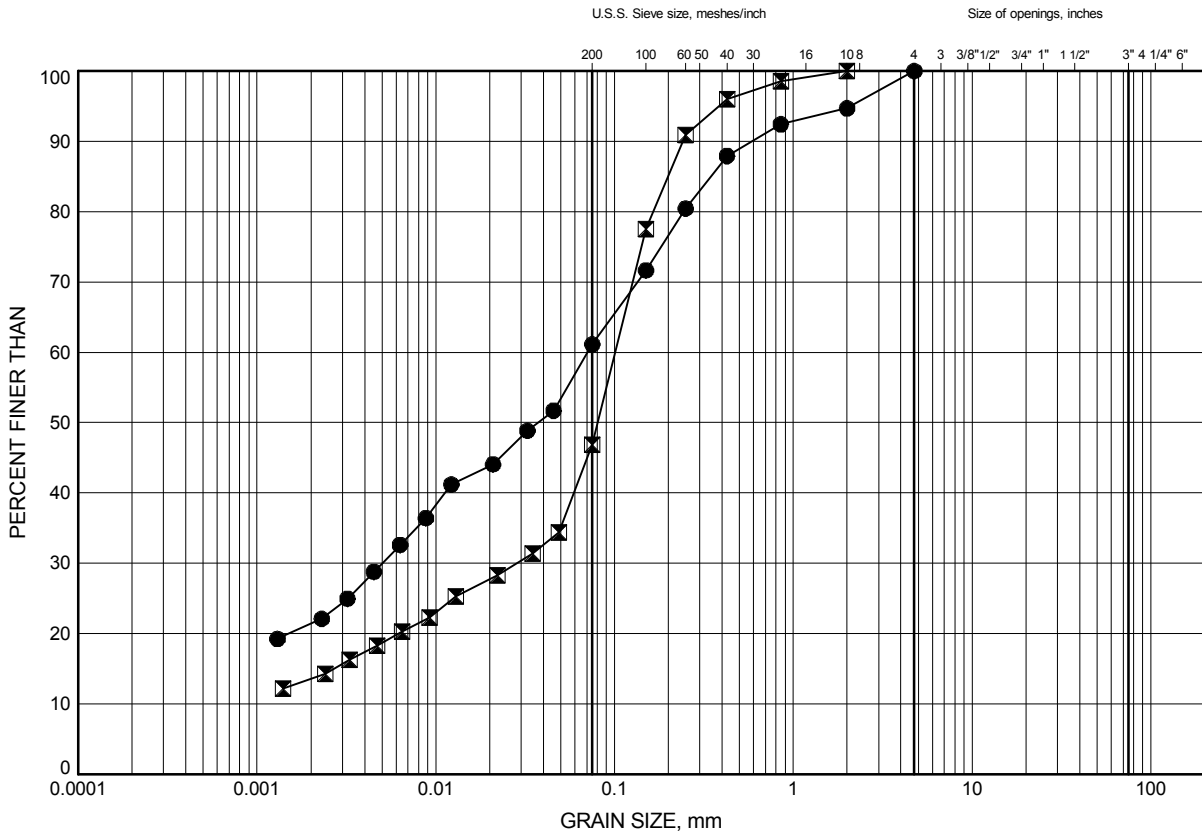
Laboratory Test Results

19-4406-6

Part B: Structural Culvert Replacement
GRAIN SIZE DISTRIBUTION

FIGURE 1

Silty Sand / Sandy Clay



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	B-1	0.30	74.19
⊠	B-2	0.30	74.59

Date April 2014
 GWP# 4012-E-0001

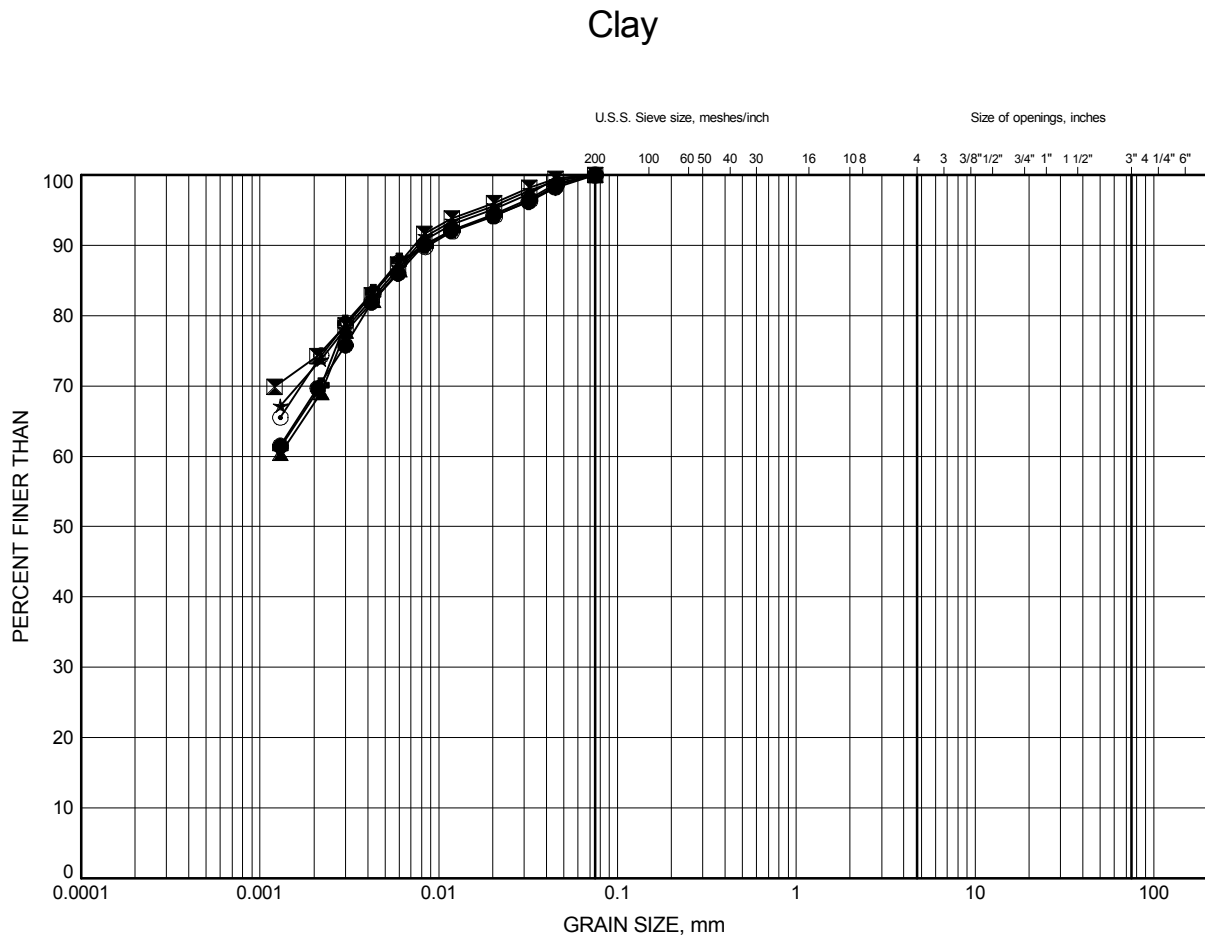


Prep'd CM
 Chkd. PC

Part B: Structural Culvert Replacement

GRAIN SIZE DISTRIBUTION

FIGURE 2



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	B-1	1.07	73.43
⊠	B-1	4.11	70.38
▲	B-2	1.83	73.07
★	B-2	3.35	71.55
⊙	B-2	9.45	65.45
⊕	B-3	1.83	72.67

Date February 2014

GWP# 4012-E-0001



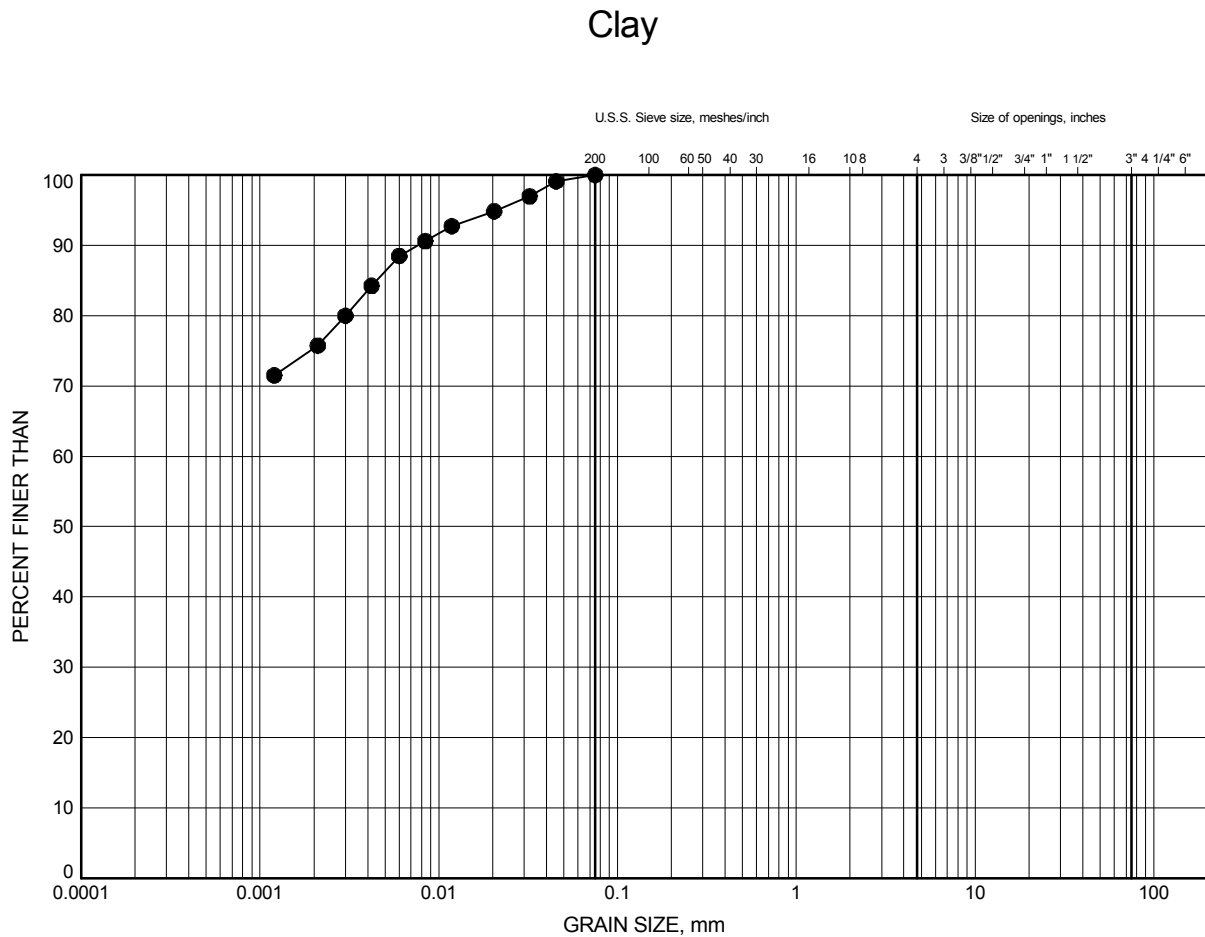
Prep'd CM

Chkd. PC

Part B: Structural Culvert Replacement

GRAIN SIZE DISTRIBUTION

FIGURE 3



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	B-3	4.88	69.62

Date February 2014
GWP# 4012-E-0001

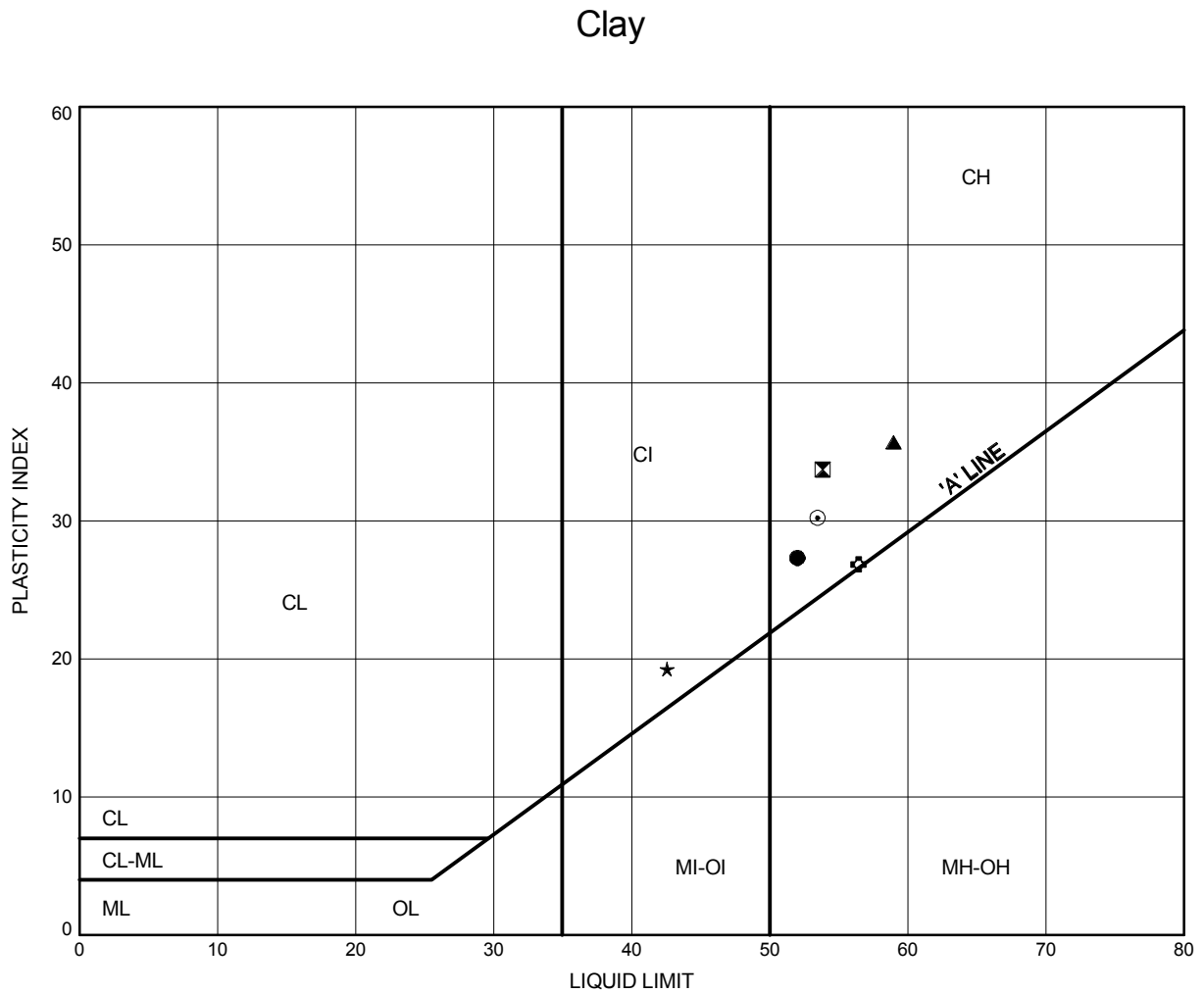


Prep'd CM
Chkd. PC

Part B: Structural Culvert Replacement

ATTERBERG LIMITS TEST RESULTS

FIGURE 4



LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	B-1	4.11	70.38
⊠	B-2	1.83	73.07
▲	B-2	3.35	71.55
★	B-2	9.45	65.45
⊙	B-3	1.83	72.67
⊕	B-3	4.88	69.62

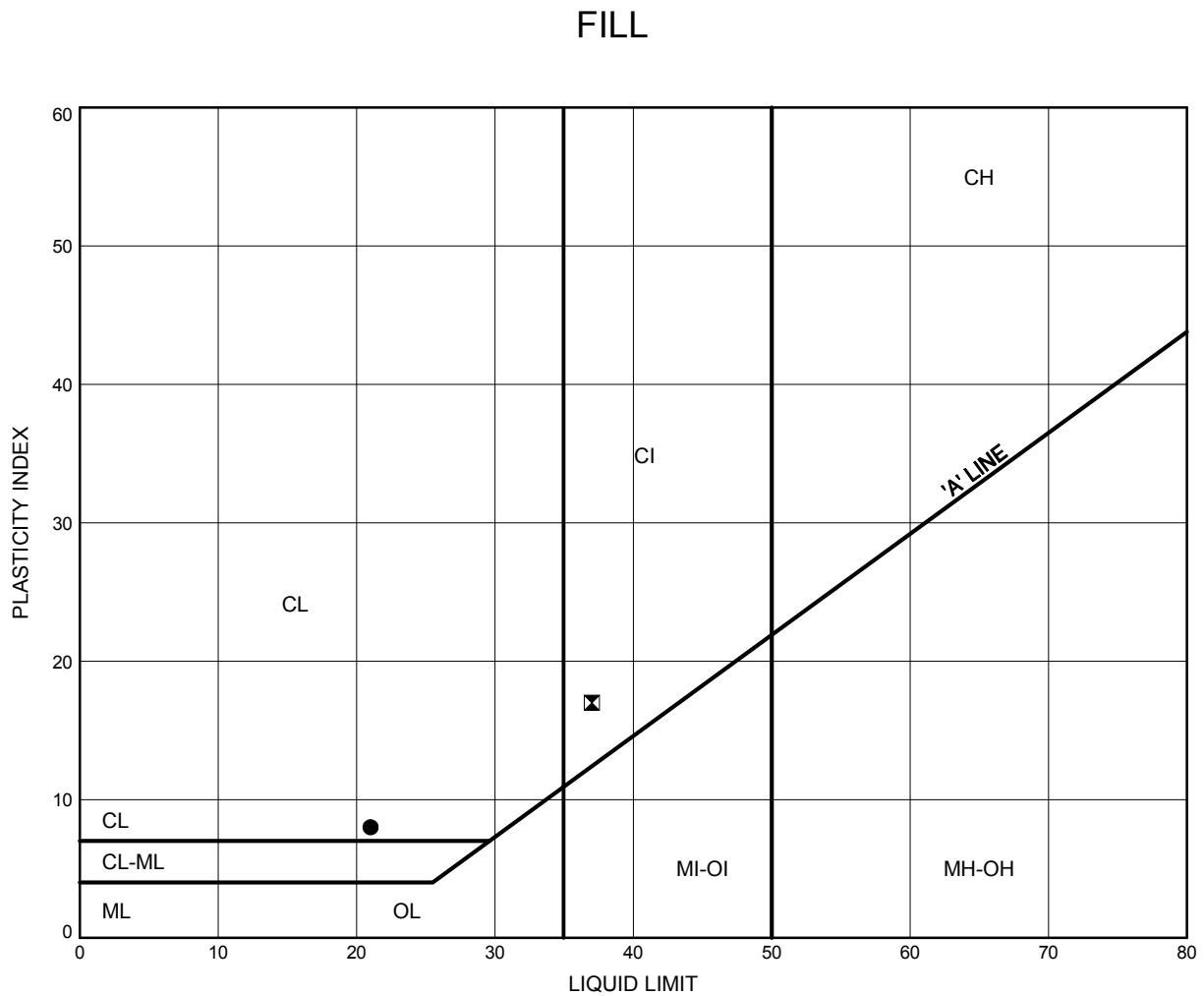
Date February 2014
GWP# 4012-E-0001



Prep'd CM
Chkd. PC

ATTERBERG LIMITS TEST RESULTS

FIGURE 5



LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	14-1a	0.30	74.93
⊠	14-1a	0.91	74.32

Date September 2014

GWP# 4012-E-0001



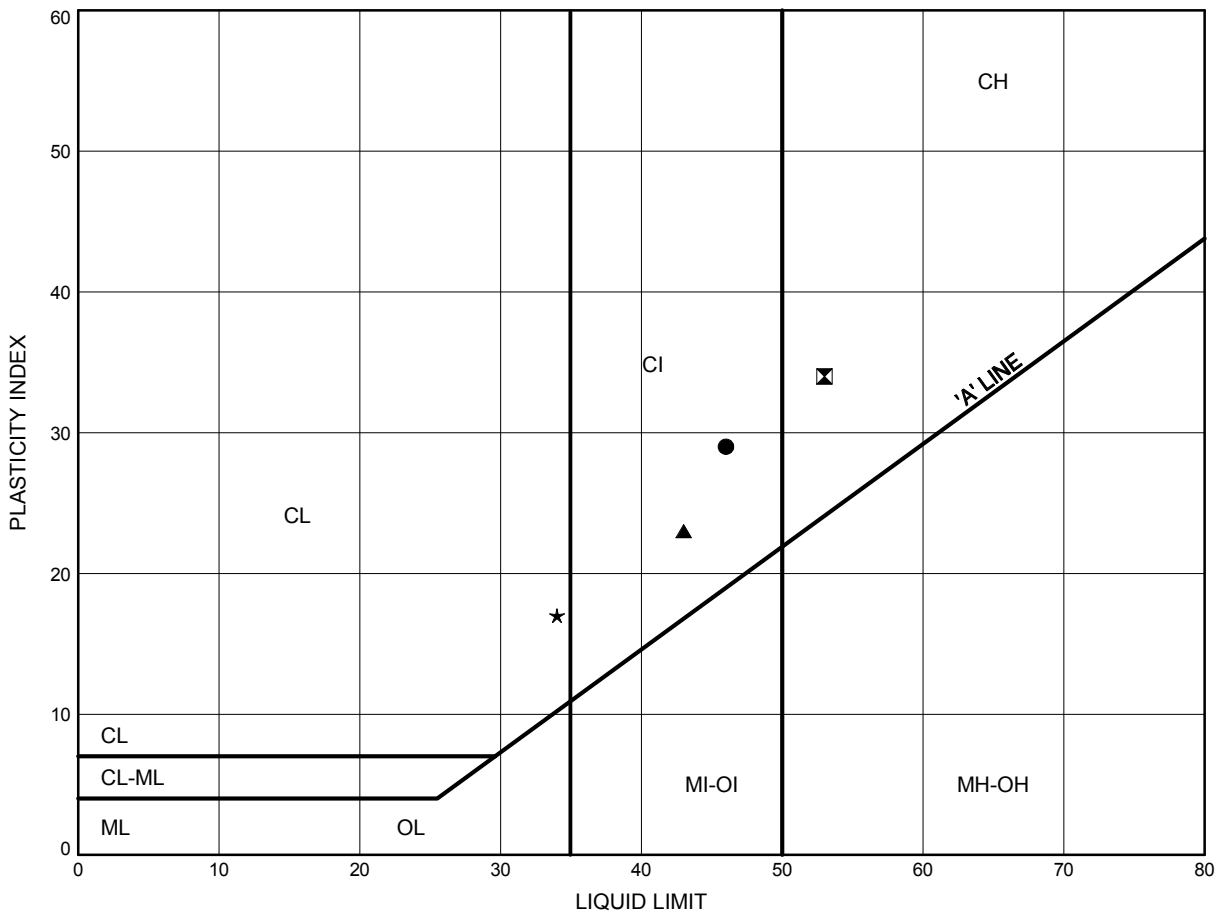
Prep'd GM

Chkd. FJG

ATTERBERG LIMITS TEST RESULTS

FIGURE 6

Silty CLAY



LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	14-1a	2.13	73.10
⊠	14-2a	1.83	72.77
▲	14-3a	1.07	74.09
★	14-3a	1.83	73.33

Date September 2014

GWP# 4012-E-0001



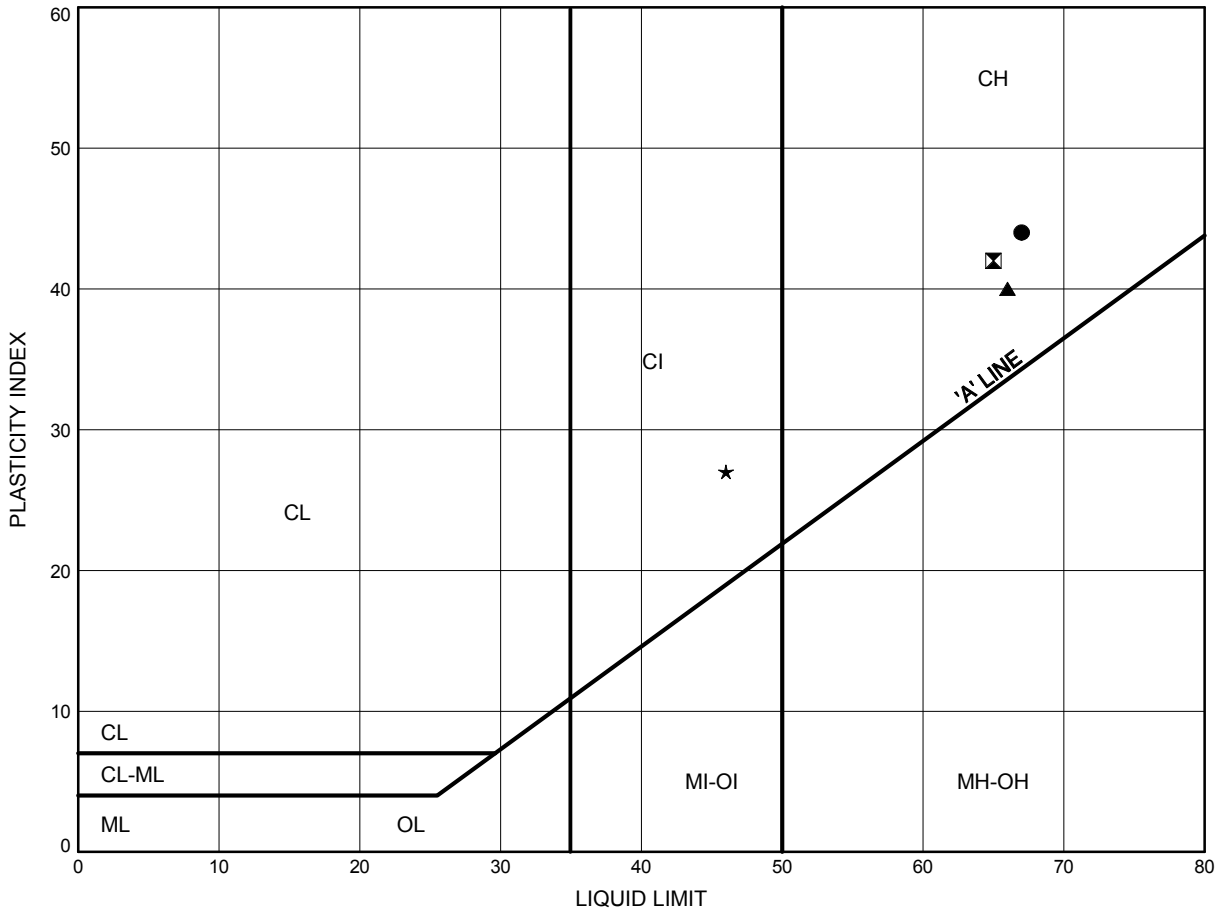
Prep'd GM

Chkd. FJG

ATTERBERG LIMITS TEST RESULTS

FIGURE 7

CLAY



LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	14-1a	5.64	69.59
⊠	14-2a	5.64	68.96
▲	14-3a	4.88	70.28
★	14-3a	9.45	65.71

Date September 2014

GWP# 4012-E-0001



Prep'd GM

Chkd. FJG

Appendix D

Selected Photographs of Culvert Locations

19-4406-6



Photo 1: South end – existing culvert inlet.



Photo 2: North end – existing culvert outlet.

Client: URS Canada Inc.

File No. 19-4406-6

E File: h:\projects\19\4406\6 - hwy 417 ottawa\foundation\culvert\report\draft\appendices\d\site photos.docx

Appendix E

Foundation Alternatives Comparisons

19-4406-6

COMPARISON OF CULVERT FOUNDATION ALTERNATIVES

Comment	Concrete - Open Footing Culvert on Clay	Concrete - Open Footing on Granular Pads	Concrete - Open Footing on Deep Foundations	Concrete Box (closed) Culvert
<i>Advantages</i>	NA	Granular pad would distribute loads and allow for use of open footings	High geotechnical resistance	Quick installation procedure due to use of pre-cast sections Loads are distributed over wide area
<i>Disadvantages</i>	Soft clay offers very low bearing resistance. Not considered feasible.	Requires deeper excavation		
<i>Risks/Consequences</i>	NA	Increases potential dewatering problems and increases costs for support systems		
<i>Relative Cost</i>	NA	Moderate cost	Very high cost	Lowest cost alternative
	NOT FEASIBLE	NOT RECOMMENDED	NOT RECOMMENDED	RECOMMENDED



COMPARISON OF WING WALL FOUNDATION ALTERNATIVES

Comment	Gabion Wall on Granular Pads	RSS Wall on Granular Pad	Concrete Wall on Deep Foundations	Extension of Concrete Box (closed) Culvert, 2H:1V Embankment slopes
Advantages	Granular pad would distribute loads Gabions are flexible and allow more settlement than more rigid structures	Granular pad would distribute loads	High geotechnical resistance	Quick installation procedure due to use of pre-cast sections Loads are distributed over wide area
Disadvantages	Very low geotechnical resistance available at ULS and SLS	Very low geotechnical resistance available at ULS and SLS		Longer culvert
Risks/ Consequences	Construction challenges due to possible excavation basal heave, construction dewatering Possible conflict during excavation with the existing culvert to the west on the south side.	Construction challenges due to possible excavation basal heave, construction dewatering Possible conflict during excavation with the existing culvert to the west on the south side.		
Relative Cost	Low cost	Low cost	Very high cost	Moderate cost
	NOT RECOMMENDED	NOT RECOMMENDED	NOT RECOMMENDED	RECOMMENDED

