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REPORT ON

GEOTECHNICAL INVESTIGATION
PROPOSED CARDINAL CREEK CULVERT
REHABILITATION / REPLACEMENT
OR174 1.3 KILOMETRES EAST OF TRIM ROAD
OTTAWA, ONTARIO

Submitted to:

GENIVAR Inc.
221-39 Robertson Road,
Ottawa, Ontario
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October 19, 2011 Our ref: 11-304-1

GENIVAR Inc.
221-39 Robertson Road,
Ottawa, Ontario
K2H 8R2

Attention: Mr. Felix Wasiewicz, P.Eng.

RE: GEOTECHNICAL INVESTIGATION
PROPOSED CARDINAL CREEK CULVERT
REHABILITATION / REPLACEMENT
OR174 1.3 KILOMETRES EAST OF TRIM ROAD
OTTAWA, ONTARIO

Dear Sir:

This report presents the results of a geotechnical investigation carried out for the existing cast in place reinforced concrete box culvert underlying OR174 1.3 kilometres east of the intersection with Trim Road in Ottawa, Ontario.

The purpose of the investigation was to identify the general subsurface conditions at the location of the culvert by means of a limited number of boreholes and, based on the factual information obtained, to provide engineering guidelines on the geotechnical design aspects of the project, including construction considerations that could influence design decisions.

PROJECT AND SITE DESCRIPTION

Project Description

Consideration is being given to rehabilitating or replacing the existing cast in place concrete box culvert underlying OR174 in Ottawa (see Key Plan, Figure 1).

The culvert is located at a distance of about 1.3 kilometres east of the intersection of Trim Road and OR174. Based on the information provided to Houle Chevrier Engineering Ltd. it is understood that the culvert was constructed in 1935 and was subsequently rehabilitated in 1955. The culvert has a span of about 5.9 metres and a length of about 45.5 metres.

It is understood that the existing culvert is located within a section of OR174 which may be widened in the future. In the vicinity of the culvert OR174 currently consists of a two lane asphaltic concrete surfaced roadway with gravel shoulders. In this area the existing roadway is aligned generally parallel with the Ottawa River (i.e. east-west) and is in a fill section which passes through areas of wetland, farmland and tree and grass covered areas. The culvert is generally aligned on a north-south axis. The difference in elevation between the roadway surface and the top of the culvert is about 3.7 metres.

There is some discrepancy with regard to the name of the creek through which the culvert flows. On-site the culvert is sign posted as “Leonard Creek”. In the General Terms of Reference provided by the City of Ottawa for this assignment and the City of Ottawa Structure Inspection Report the structure is referred to as the OR174 Bridge Culvert over Cardinal Creek (SN897240). For clarity, in accordance with the City of Ottawa documentation the culvert will be referred to in this report as the Cardinal Creek culvert.

Review of Geology Maps

Existing geology maps of the Ottawa area indicate that the near surface overburden in the vicinity of the Cardinal Creek culvert consists of organic soils. Erosional terraces through marine silts and clays are also present in the area. Bedrock geology maps indicate that the soils at the site are underlain by dolostone of the Oxford formation at depths ranging from 10 to 15 metres.

Fill material associated with previous development on the site should also be anticipated.

SUBSURFACE INVESTIGATION

The field work for this investigation was carried out on June 29, 2011. At that time, two (2) boreholes numbered 11-1 and 11-2 were advanced using portable drilling equipment owned and operated by OGS Drill of Almonte, Ontario. The boreholes were advanced to depths of about 6.1 to 7.3 metres below ground surface at the inlet and outlet of the existing culvert.

Standard penetration testing was carried out in the boreholes at regular intervals of depth and samples of the soils encountered were recovered using drive open sampling equipment. In-situ vane shear strength testing was carried out where possible in the clayey deposits to measure the undrained shear strength. The field work was supervised by a member of our engineering staff.

Following completion of the drilling, the soil samples were returned to our laboratory for examination by the project engineer and for laboratory testing. One (1) sample of the creek water was sent to Exova Laboratories Ltd. to assess the corrosive potential of the water in the creek on exposed steel and concrete. In addition, one (1) soil sample was sent to Paracel Laboratories Ltd. to assess the sulphate content in the soil. The results of the chemical analysis on the creek sample are provided in Attachment A. The results of the chemical analysis on the soil sample are provided in Attachment B.

Descriptions of the subsurface conditions logged in the boreholes are provided on the Record of Borehole sheets following the text of this report. The borehole locations were surveyed by Houle Chevrier Engineering personnel using our Trimble global positioning equipment and are provided on the Site Plan, Figure 2. The ground surface elevations at the location of the boreholes were measured relative to Geodetic datum, to an accuracy of ± 20 millimetres.

SUBSURFACE CONDITIONS

General

As previously indicated, the soil conditions logged in the boreholes are given on the Record of Borehole sheets following the text of this report. The logs indicate the subsurface conditions at the specific test locations only. Boundaries between zones on the logs are often not distinct, but rather are transitional and have been interpreted. Subsurface conditions at other than the borehole locations may vary from the conditions encountered at the test locations. In addition to soil variability, fill of variable physical and chemical composition can be present over portions of the site.

The soil descriptions in this report are based on commonly accepted methods of classification and identification employed in geotechnical practice. Classification and identification of soil involves judgement and Houle Chevrier Engineering Ltd. does not guarantee descriptions as exact, but infers accuracy to the extent that is common in current geotechnical practice.

The following presents an overview of the subsurface conditions encountered in the boreholes advanced during this investigation.

Topsoil Fill

A surficial layer of topsoil fill material was encountered in boreholes 11-1 and 11-2. The thickness of the topsoil fill ranges from about 100 to 150 millimetres.

Fill Material

Fill material was encountered beneath the topsoil fill in all of the boreholes. The depth to the underside of the fill material ranges from about 1.8 to 2.7 metres below ground surface.

The fill material is variable in colour and consistency but can generally be described as grey brown and grey silty clay, with some gravel. The fill material was observed to contain cobbles and boulders, organic material and refuse.

Standard penetration tests carried out within the fill material gave N values in the range of 1 to 6 blows per 0.3 metres of penetration. It should be noted that the higher N values may be due to the presence of cobble and boulder sized obstructions within the fill material.

Possible Former Topsoil Layer

A layer of possible former topsoil was encountered beneath the fill material in borehole 11-2. The thickness of the former topsoil layer is about 150 millimetres.

Silty Clay

A deposit of grey silty clay of marine origin was encountered beneath the fill material in borehole 11-1 and beneath the possible former topsoil layer in borehole 11-2.
Standard penetration tests carried out within the silty clay gave N values in the range of 3 to 10 blows per 0.3 metres of penetration. In-situ vane shear strength tests carried out within the silty

clay gave undrained shear strengths ranging from 62 to 93 kilopascals, which indicates a stiff consistency. The remoulded vane shear test values generally range from 16 to 29 kilopascals, which reflect the effect of disturbance on the strength of the silty clay deposit.

Boreholes 11-1 and 11-2 were terminated within the silty clay at depths ranging from 6.1 to 7.3 metres below ground surface.

Groundwater Conditions

Standpipe piezometers were not installed as part of this geotechnical investigation. However, samples of the soils recovered during our ground investigation were generally wet below a depth of about 0 to 1.3 metres below ground surface on June 29, 2011.

The top of the culvert was surveyed at 42.8 metres Geodetic, on the north side of Cardinal Creek and between about 42.9 and 43.0 metres Geodetic on the south side of the creek. The elevation of the surface of the water within the culvert was at the about 1.4 metres below the top of the culvert, or at about 41.5 metres Geodetic datum on July 21, 2011. As previously stated the elevations were measured using our Trimble GPS equipment which has an accuracy of ± 20 millimetres.

It should be noted that the groundwater level could be higher during wet periods of the year, such as the early spring, or following periods of heavy precipitation or snow melt. Also, the surface water conditions in the creek will likely affect the groundwater levels.

Groundwater and Surface Water Chemistry Relating to Corrosion

The chemical testing on a water sample recovered from the creek showed the following results (see Attachment A):

Test Item	***Water Sample***
Conductivity (micromhos/centimetre)	715
pH	8.2
Sulphate Content (mg/L)	57
Chloride Content (mg/L)	67
Hardness (mg/L)	211
Calcium (mg/L)	50
Magnesium (mg/L)	21

The chemical testing on a sample of soil recovered from borehole 11-1 at a depth of about 1.8 metres below ground surface showed the following results (see Attachment B):

Test Item	***Borehole 11-1***
Conductivity (micromhos/centimetre)	1149
pH	7.1
Sulphate Content (ug/g)	116
Chloride Content (ug/g)	812

PROPOSED CULVERT REHABILITATION / REPLACEMENT

General

This section of the report provides engineering guidelines on the geotechnical design aspects of the project based on our interpretation of the borehole information and project requirements. It is stressed that the information in the following sections is provided for the guidance of the design engineers and is intended for this project only. Contractors bidding on or undertaking the works should examine the factual results of the investigation, satisfy themselves as to the adequacy of the information for construction, and make their own interpretation of the factual data as it affects their construction techniques, schedule, safety and equipment capabilities.

The professional services retained for this project include only the geotechnical aspects of the subsurface conditions at this site. The presence or implications of possible surface and/or subsurface contamination resulting from previous uses or activities of this site or adjacent properties, and/or resulting from the introduction onto the site from materials from off site sources are outside the terms of reference for this geotechnical report.

Proposed Construction Alternatives

It is understood that consideration is being given to either rehabilitating or replacing the existing culvert. Based on our discussions with Genivar Inc. it is understood that the following options are being considered:

1) Culvert Repair and Modification;
2) Culvert Replacement (Cast in Place or Precast Concrete);
3) Partial Demolition and Replacement;
4) Replacement of the Culvert with a Bridge Structure.

The geotechnical aspects associated with these options are discussed in the following sections.

Option 1 : Culvert Repair and Modification

General

This option would include repairing the existing culvert and constructing retaining walls to facilitate future road widening works. It should be noted that the culvert repair is not considered within our area of professional expertise and, as such, this aspect of the construction option has not been considered in the report. Furthermore, the details of the proposed road widening and retaining walls were not available to Houle Chevrier Engineering Ltd. at the time of writing this report. The following sections discuss the construction of the retaining walls, from a geotechnical perspective.

Excavation

Excavation for the retaining walls will be carried out though fill material associated with the existing roadway embankment, former topsoil and silty clay. The sides of the excavation should be sloped in accordance with the requirements in Ontario Regulation 213/91 under the Ontario Occupation Health and Safety Act. According to the act, the embankment fill and upper portion of the silty clay can be classified as Type 3 soils. As such, allowance should be made for 1 horizontal to 1 vertical, or flatter, excavation side slopes for excavations up to 3 metres below the natural ground surface level.

Groundwater inflow from the fill and overburden deposits should be controlled by pumping from sumps with the excavation. Cofferdams should be constructed around the area of excavation using compacted, relatively impermeable earth fill material (such as compacted, weathered silty clay) or steel sheeting, and the creek flow should be pumped or diverted during the construction. The presence of rock fill material within the embankment may preclude the use of steel sheeting without prior excavation of the rock fill. The groundwater handling should be carried out in accordance with Provincial and local regulations. To reduce the groundwater / surface water pumping requirements, we suggest that the excavation be planned for the dry period of the year (i.e. June to September). Insofar as groundwater pumping is concerned, it is possible that the amount of groundwater infiltration at this site could exceed 50,000 litres per day. Therefore, we recommend that an Ontario Ministry of the Environment (MOE) Permit to Take Water (PTTW) be obtained in advance of the construction. It is our experience that a PTTW takes about 3 months to obtain from the time of application. The groundwater disposal should be carried out in accordance with Provincial and local regulations.

The contractor should be required to prepare and submit an excavation and groundwater management plan for review and approval as part of the contract.

Retaining Wall Foundations

Based on the results of this investigation, it is considered that the propose retaining walls could be founded on spread footings bearing on or within the native deposit of silty clay, or on a pad of engineered fill material above the native deposits. The fill material and former topsoil as well as any deposits of peat, alluvium or other recent and/or organic deposits which may be encountered at the foundation level are not considered suitable for the support of the proposed retaining walls and should be removed from the foundation area.

In areas where the underside of footing level is above the level of the native soil or where subexcavation of soil is required, the grade below the proposed footings could be raised with compacted granular material (engineered fill). The engineered fill should consist of granular material meeting Ontario Provincial Standard Specification (OPSS) requirements for Granular B Type II materials. OPSS documents allow recycled asphaltic concrete to be used as Granular B Type II material. Since the source of recycled material cannot be determined, it is suggested that any granular materials used beneath the proposed footings be composed of virgin material only. The granular material should be compacted in maximum 200 millimetre thick lifts to at least 98 percent of the standard Proctor dry density value.

Spread footing bearing on the native, undisturbed overburden deposits should be sized using a net geotechnical reaction at Serviceability Limit State (SLS) of 100 kilopascals and a factored resistance at Ultimate Limit State (ULS) of 250 kilopascals. Spread footings bearing on a pad of engineered fill above the native deposits described in this report should be sized using a geotechnical reaction at Serviceability Limit State (SLS) of 150 kilopascals and a factored geotechnical resistance at Ultimate Limit State (ULS) of 300 kilopascals, provided the thickness of the pad is at least 1.5 times the width of the footings. It should be noted that these bearing pressures do not include the weight of the footing.

The post construction total and differential settlement of footings should be less than 25 and 15 millimetres, respectively, provided that all loose or disturbed soil is removed from the bearing surfaces.

Frost Protection Requirements for Foundations

The footings should be provided with at least 1.8 metres of earth cover for frost protection purposes. For footings which are bearing on engineered fill material, the required frost cover could be reduced by the thickness of the engineered fill.

Alternatively, the required frost protection could be provided by means of a combination of earth cover and extruded polystyrene insulation. An insulation detail could be provided upon request.

Retaining Wall Backfill

The backfill to the retaining walls should consist of free draining, non-frost susceptible sand and gravel, such as that meeting OPSS requirements for Granular B Type I or II. The backfill behind the retaining walls should extend at least 1.8 metres horizontally from the interior face of the walls. The backfill should be compacted in maximum 200 millimetre thick lifts to at least 95 percent of the standard Proctor dry density value using suitable hand operated vibratory compaction equipment.

The lateral pressure on the retaining walls due to the earth loads will depend on the amount of movement that occurs at the top of the walls as a result of the earth pressures. In accordance with the CHBDC, an "active" earth pressure coefficient, should be used to calculate the horizontal loads on the retaining wall if the lateral movement at the top of the retaining wall is more than about 0.2 percent of the wall height. For the case of a rigid wall (i.e., no wall movement), an "at rest" earth coefficient should be used. A linear interpolation between the pressures due to the backfill can be used if the wall movement falls between the "at rest" and the "active" cases.

As such the horizontal earth pressure on the walls should be calculated using:

$$P = K\gamma H$$

Where;

P: Active or at rest earth pressure at depth H (kilopascals)
γ: Moist material unit weight (22 kN/m^3)
K: "At rest" or "Active" earth pressure coefficient (0.35 or 0.22, respectively)
H: Depth below ground surface (metres)

To prevent buildup of hydrostatic pressure behind the retaining walls, the wall backfill material should be drained with a perforated pipe, which outlets by gravity to a suitable outlet.

In accordance with the CHBDC, a compaction induced surcharge pressure should be included. The magnitude of the compaction surcharge pressure depends on the mass and type of compaction equipment. For light, hand operated compaction equipment having a mass of approximately 400 kilograms, the surcharge pressure can be taken as 16 kilopascals. The surcharge pressure should be increased if heavier equipment is used.

Seismic shaking can increase the forces on the retaining wall during or following an earthquake. The increase in pressure may be estimated using the method suggested by Mononobe and Okabe (refer to section C4.6.4 of the CHBDC). For non-yielding retaining walls, which are restrained against movement, and assuming that the vertical acceleration coefficient is 2/3 of the horizontal acceleration, the combined coefficient of static and seismic active earth pressure, K_{AE}, on the back of the retaining walls can be calculated as 0.48. If some outward displacement of the retaining walls can occur, the earth pressure under earthquake conditions will be much less. For example, assuming an outward displacement of about 50 millimetres, the combined coefficient will be about 0.30. Further guidelines the site specific earth pressures can be provided by us at the final design stage.

Option 2 : Culvert Replacement (Cast in Place or Precast Concrete Sections)

General

It is understood that this construction alternative would involve excavation and removal of the existing cast in place culvert and installation of either a cast in place concrete, or a precast concrete culvert with retaining walls. The culvert could be made longer than the existing culvert (to facilitate the future road widening) or could be constructed with retaining walls. Our guidelines for the geotechnical aspects associated with the construction of retaining walls have been covered in Option 1. The following sections discuss the construction of either a lengthened cast in place concrete or lengthened precast concrete box culvert, from a geotechnical perspective.

Excavation

It is understood that the proposed elevation of the base of the culvert would be about elevation 39.3 metres. To allow excavation and removal of the existing structure, the excavation for the proposed culvert will be carried out through asphaltic concrete, roadway granular material, fill material associated with the roadway embankment (which may contain significant quantites of boulder size fragments of rock) and possibly silty clay. The existing reinforced concrete structure will have to be demolished and removed from the site.

The sides of the excavation should be sloped in accordance with the requirements in Ontario Regulation 213/91 under the Ontario Occupation Health and Safety Act. According to the act, the embankment fill and upper portion of the silty clay can be classified as Type 3 soils. As such, allowance should be made for 1 horizontal to 1 vertical, or flatter, excavation side slopes for excavations up to 3 metres below the natural ground surface level.

The following alternatives could be considered from an excavation point of view:

1) Close the existing roadway on a short term basis and construct the entire culvert in one stage; OR
2) Install a temporary roadway detour in conjunction with a temporary shoring system (if necessary), and thereby allowing the roadway to remain in use.

The cost for a detour (and a temporary shoring system) would significantly increase both the cost of excavation and the time to carry out the work.

Groundwater inflow from the fill and overburden deposits should be controlled by pumping from sumps with the excavation. Cofferdams should be constructed around the area of excavation using compacted, relatively impermeable earth fill material (such as compacted, weathered silty clay) or steel sheeting, and the creek flow should be pumped or diverted during the construction. The presence of rock fill material within the embankment may preclude the use of steel sheet piling without prior excavation of the rock fill. The groundwater handling should be carried out in accordance with Provincial and local regulations. To reduce the groundwater / surface water pumping requirements, we suggest that the excavation be planned for the dry period of the year (i.e. June to September). Insofar as groundwater pumping is concerned, it is possible that the amount of groundwater infiltration at this site could exceed 50,000 litres per day. Therefore, we recommend that an Ontario Ministry of the Environment (MOE) Permit to Take Water (PTTW) be obtained in advance of the construction. It is our experience that a PTTW takes about 3 months to obtain from the time of application. The groundwater disposal should be carried out in accordance with Provincial and local regulations.

The contractor should be required to prepare and submit an excavation and groundwater management plan for review and approval as part of the contract.

Culvert Bedding

Once excavation of the culvert structure is completed, all fill loose / soft, organic or disturbed/water softened soil should be removed from the subgrade surface. If necessary, the grade below the proposed structure could be raised with imported granular material conforming to OPSS Granular B Type II. To provide adequate spread of load below the structure, any granular material placed should extend at least 0.3 metres horizontally beyond the edge of the structure and down and out from this point at 1 horizontal to 1 vertical, or flatter.

The bedding layer should be composed of at least 150 millimetres of OPSS Granular A and should be placed in accordance with OPSD 802.010 for Type 3 soils. The Granular A should be compacted in maximum 200 millimetre thick lifts to at least 95 percent of the standard Proctor dry density value. Jumping jack type compaction equipment could cause disturbance to the sensitive silty clay and should not be used.

The vertical load on the structure due to the roadway pavement structure can be calculated using a material unit weight of 22 kilonewtons per cubic metre. The live load effect from traffic should be considered in the design.

A net geotechnical reaction at Serviceability Limit State (SLS) of 150 kilopascals and a factored resistance at Ultimate Limit State (ULS) of 250 kilopascals could be assigned to the native deposits of silty clay at the proposed elevation of the base of the culvert (i.e. about 39.3 metres, Geodetic).

The silty clay deposit at this site is sensitive to disturbance from ponded water, vibration and construction traffic. As such, it is suggested that if the excavation for the culvert is carried out in open cut, the final trimming to subgrade level be carried out using a hydraulic shovel equipped with a flat bucket. Allowance should be made to place a 300 millimetre (minimum) thick subbedding layer composed of OPSS Granular B Type II and a woven geotextile separator (such as Thrace-Linq GTF-200) over the subgrade surface immediately after excavation and removal of any disturbed soil. Any fill material below the proposed culvert should be removed and replaced with OPSS Granular B Type II. The granular material should be compacted to at least 95 percent of the standard Proctor dry density vale using suitable compaction equipment. A static drum roller may be required in order to limit vibrations and thereby reduce the potential for disturbance to the subgrade.

Culvert Backfill

The concrete box culvert should be backfilled with free draining, non-frost susceptible sand or sand and gravel, such as that meeting OPSS requirement for Granular B Type I or II. The backfill should extend at least 1.2 metres horizontally from the exposed face of the culvert walls. The backfill should be compacted in maximum 200 millimetre thick lifts to at least 95 percent of the standard Proctor dry density value using suitable, walk behind vibratory compaction equipment.

The horizontal earth pressures on the culvert walls should be calculated using:

$$P_o = K_o \gamma H$$

where,

P_o: At rest earth pressure at depth H (kilopascals)
γ: Moist material unit weight, 22 kilonewtons per cubic metre (kN/m^3)
K_o: At rest earth pressure coefficient, 0.35
H: Depth below ground surface (metres)

The lateral pressures due to compaction should be considered in the design. The magnitude of the compaction surcharge pressure depends on the mass and type of compaction equipment. For light, walk behind, compaction equipment having a mass of approximately 400 kilograms, the surcharge pressure can be taken as 16 kilopascals. The surcharge pressure should be increased if heavier equipment is used. Compaction surcharge pressures should be considered for both the structural and geotechnical designs for the "at rest" earth pressure case.

The vertical load on the culvert can be calculated using a material unit weight of 22 kilonewtons per cubic metre. The live load effects from traffic should be considered in the design.

Frost Protection of Culvert

The frost protection requirements below a closed concrete box structure depend on the expected water conditions in the creek during the winter period. If the water flow is continuous throughout the winter period, frost protection is likely not required, since flowing water will prevent freezing below the creek bottom. If the water flow is expected to be intermittent during the winter period or the water is lower during the winter period and freezes throughout, the bedding thickness described in the previous section of this report may not be sufficient to provide frost protection, which could result in heaving of the bottom of the structure. In this case it is suggested that the subgrade surface be protected with a combination of earth cover and extruded polystyrene insulation. An insulation detail could be provided upon request.

Erosion Protection

The inlet and outlet of the culvert should be protected from erosion using rip rap. For water flows of up to 3 metres per second, the rip rap should consist of well graded crushed rock with particle sizes ranging from about 100 to 180 millimetres and a mean particle size of at least 150 millimetres. The rip rap should meet the requirements in OPSS 1004 and should be placed in accordance with OPSS 511. For rip rap having a maximum particle size of about 180 millimetres, the thickness of the rip rap should be at least 300 millimetres (i.e. 1.5 times the maximum particle size). A nonwoven geotextile (for example Linq 180EX, or approved equivalent) separator should be placed between the subgrade surface and the rip rap. All seams in the geotextile should be sewn or provided with at least 0.5 metres of overlap.

Embankment and Roadway Reinstatement

Following compaction of the backfill material the embankment and roadway should be reinstated.

The embankment could be reinstated using material meeting OPSS Granular B Type II requirements, Select Subgrade Material, or well shattered and graded rock fill material. In low, wet areas such as should be anticipated at a culvert location well shattered and graded rock fill material is preferred.

The Select Subgrade Material or Granular B Type II should be placed in maximum 300 millimetre thick lifts and compacted to at least 95 percent of the standard Proctor dry density value usung vibratory compaction equipment. Rock fill should also be placed in thin lifts and suitably compacted either with a large drum roller, the haulage and spreading equipment, or a combination of both. Prior to placing granular material for the roadway, the exposed subgrade should be heavily proof rolled under dry conditions and inspected and approved by geotechnical personnel. Any soft / loose areas evident from the proof rolling should be subexcavated and replaced with suitable fill approved by the geotechnical engineer.

This section of the OR174 is maintained by the City of Ottawa. Traffic data was not available for the roadway at the time of this report; however, the roadway could be classified as a major arterial. As such, we suggest that the following pavement structure be used:

> One 40 to 50 millimetre layer of Superpave 12.5 FC1 or FC2 (Traffic Level D) over;
> Two 50 millimetre layer of Superpave 19.0 (Traffic Level D) over;
> 150 millmetres of OPSS Granular A, over
> 600 millimetres of OPSS Granular B Type II

Performance grade PG 58-34 asphaltic concrete should be specified in accordance with City of Ottawa standards.

Asphaltic concrete and granular tapers should be provided, if required, where the new pavement will abut the existing pavement. The granular tapers should be sloped at 10 horizontal to 1 vertical.

All imported granular materials should be placed in maximum 200 millimetre thick lifts and should be compacted to at least 98 percent of the standard Proctor dry density value using suitable vibratory compaction equipment.

The City of Ottawa requires that 40 to 50 millimetres of asphaltic concrete be milled to a distance of 300 millimetres on either side of excavations and be replaced with the HL3 surface course. This approach has been found by the City to reduce the potential for reflective cracking and could be considered for the cut within the paved roadway on this project.

Option 3 : Partial Demolition and Replacement

General

It is understood that this construction alternative would involve excavation of the existing embankment fill material, construction of spread footings behind the culvert, demolition of the top and a portion of the side walls of the existing culvert and installation of a precast frame units (open box culvert). The geotechnical aspects associated with this construction option are provided in the following sections.

Excavation

It is understood that as part of this proposed construction option the existing culvert will be partially demolished leaving the existing culvert base and a portion of the side walls in place.

To allow excavation and removal of the existing structure, the excavation for the proposed culvert will be carried out through asphaltic concrete, roadway granular material, fill material associated with the roadway embankment and possibly silty clay.

Similar alternatives with regard to traffic management are considered applicable to Option 3 as were presented for Option 2.

Groundwater inflow from the fill and overburden deposits should be controlled by pumping from sumps with the excavation. It may be possible to maintain the culvert flow within the partly demolished culvert section during construction of the replacement culvert. Where this is not possible (or desirable) cofferdams should be constructed around the area of excavation as described previously for Option 1.

The contractor should be required to prepare and submit an excavation and groundwater management plan for review and approval as part of the contract.

Spread Footing Design

Based on the results of this investigation, it is considered that the proposed culvert foundations could be founded on spread footings bearing on or within the native deposit of silty clay, or on a pad of engineered fill material above the native deposits. The fill material and former topsoil as well as any deposits of peat, alluvium or other recent and/or organic deposits which may be encountered at the foundation level are not considered suitable for the support of the proposed structure and should be removed from the foundation area.

In areas where the underside of footing level is above the level of the native soil or where subexcavation of soil is required, the grade below the proposed footings could be raised with compacted granular material (engineered fill). In this case, to prevent transfer of load between the footings and pad of granular fill material to the partially demolished culvert it is recommended that the footings be located beyond a line extending upwards and outwards at a minimum of 1 horizontal to 1 vertical from the base of the existing culvert. The engineered fill should consist of granular material meeting Ontario Provincial Standard Specification (OPSS) requirements for Granular B Type II materials. OPSS documents allow recycled asphaltic concrete to be used as Granular B Type II material. Since the source of recycled material cannot be determined, it is suggested that any granular materials used beneath the proposed footings be composed of virgin material only. The granular material should be compacted in maximum 200 millimetre thick lifts to at least 98 percent of the standard Proctor dry density value.

Spread footing bearing on the native, undisturbed overburden deposits should be sized using a net geotechnical reaction at Serviceability Limit State (SLS) of 100 kilopascals and a factored resistance at Ultimate Limit State (ULS) of 250 kilopascals. Spread footings bearing on a pad of engineered fill above the native deposits described in this report should be sized using a geotechnical reaction at Serviceability Limit State (SLS) of 150 kilopascals and a factored geotechnical resistance at Ultimate Limit State (ULS) of 300 kilopascals, provided the thickness of the pad is at least 1.5 times the width of the footings. It should be noted that these bearing pressures do not include the weight of the footing.

The post construction total and differential settlement of footings should be less than 25 and 15 millimetres, respectively, provided that all loose or disturbed soil is removed from the bearing surfaces.

Frost Protection for Foundations

As previously stated the frost protection requirements for the proposed culvert depend on the expected water conditions in the creek during the winter period, and in this case on the anticipated channel width once the demolition works are completed.

If continuous water flow which will extend across the full width of the newly installed culvert (including the area between the partially demolished culvert walls and the newly installed culvert walls) will occur then frost protection of the footings is not considered necessary. The footing should however be placed at a depth which will provided adequate protection from scour, as described in subsequent sections of this report.

If the water flow is expected to be intermittent during the winter period, or the flow will be confined within the partially demolished culvert then 1.8 metres of earth cover should be provided to the footings. Alternatively, the footing depth could be reduced and the required frost protection achieved using a combination of earth cover and extruded polystyrene sheets (such as DOW SM, or approved equivalent). A detail for the insulation of the footings could be provided upon request.

Erosion Protection

The inlet and outlet of the culvert should be protected from erosion using rip rap. For water flows of up to 3 metres per second, the rip rap should consist of well graded crushed rock with particle sizes ranging from about 100 to at 180 millimetres and a mean particle size of at least 150 millimetres. The rip rap should meet the requirements in OPSS 1004 and should be placed in accordance with OPSS 511. For rip rap having a maximum particle size of about 180 millimtres, the thickness of the rip rap should be at least 300 millimetres (i.e. 1.5 times the maximum particle size). A nonwoven geotextile (for example Thrace-Linq 180EX, or approved equivalent) separator should be placed between the subgrade surface and the rip rap. All seams in the geotextile should be sewn or provided with at least 0.5 metres of overlap.

The details of the proposed construction methodology were not available to Houle Chevrier Engineering Ltd. at the time of writing this report. If water flow is to remain within the partially demolished culvert the requirement for erosion protection along the base of the culvert could be negated. However, if water flow is anticipated across the full width of the newly installed culvert then a minimum of 300 millimetres of rip rap should also be placed above in the channel between the partly demolished culvert and the new culvert walls.

Culvert and Foundation Backfill

The concrete box culvert should be backfilled with free draining, non frost susceptible sand or sand and gravel, such as that meeting OPSS requirements for Granular B Type I or II. The backfill should extend at least 1.2 metres horizontally from the exposed face of the culvert walls. The backfill should be compacted in maximum 200 millimetre thick lifts to at least 95 percent of the standard Proctor dry density value using suitable, walk behind vibratory compaction equipment.

The horizontal earth pressures on the culvert walls should be calculated using:

$$P_o = K_o \gamma H$$

where,

P_o: At rest earth pressure at depth H (kilopascals)
γ: Moist material unit weight, 22 kilonewtons per cubic metre (kN/m^3)
K_o: At rest earth pressure coefficient, 0.35
H: Depth below ground surface (metres)

In accordance with the CHBDC, a compaction induced surcharge pressure should be included. The magnitude of the compaction surcharge pressure depends on the mass and type of compaction equipment. For light, hand operated compaction equipment having a mass of approximately 400 kilograms, the surcharge pressure can be taken as 16 kilopascals. The surcharge pressure should be increased if heavier equipment is used.

The vertical load on the culvert can be calculated using a material unit weight of 22 kilonewtons per cubic metre. The live load effects from traffic should be considered in the design.

Embankment and Reinstatement

Refer to Option 2 for our guidelines on roadway reinstatement.

Option 4 : Replacement with a Bridge Structure

General

It may be preferential to replace the existing culvert section with a bridge structure. While the details of such a structure are not currently available to Houle Chevrier Engineering Ltd. and the current level of ground investigation is not considered sufficient to adequately address the geotechnical issues associated with bridge construction preliminary guidelines are provided in the following sections.

Excavation

As part of this proposed construction option the existing culvert would be demolished. To allow excavation and removal of the existing structure, the excavation for the proposed culvert will be carried out through asphaltic concrete, roadway granular material, fill material associated with the roadway embankment and possibly silty clay.

Similar alternatives with regard to traffic management are considered applicable to Option 4 as were presented for Options 2 and 3.

Groundwater inflow from the fill and overburden deposits should be controlled by pumping from sumps with the excavation. Cofferdams should be constructed around the area of excavation as described previously.

The contractor should be required to prepare and submit an excavation and groundwater management plan for review and approval as part of the contract.

Bridge Foundations

Based on the results of this investigation and the available geotechnical information for the area the bridge abutments should be founded on deep foundations, such as steel piles, which derive support by end bearing on or within bedrock. Steel H piles, such as HP310X79, or thick walled, concrete filled steel pipe piles would be suitable for this site.

The depth to bedrock at this site or the condition of the bedrock has not been determined. Available geology maps indicate that dolostone bedrock is present at the site at a depth of 10 to 15 metres below ground surface, which based on the current profile would suggest piles lengths of between 10 and 15 metres would be required at this site. No unusual restrictions are anticipated with supporting end bearing piles on dolostone bedrock which is of relatively good quality.

The geotechnical reaction at Serviceability Limit State (SLS) on a 245 millimetre diameter, closed ended, steel pipe pile with a wall diameter of 11 millimetres could be taken as 1050 kilonewtons. For piles driven to refusal on competent bedrock the Ultimate Limit State resistance could be taken as the factored structural capacity of the pile. This assumes that the steel has a minimum yield strength of 340 megapascals and that the pipe is filled with 30 megapascal concrete.

As a second design example, the SLS reaction on an HP310x79 pile could be taken as 850 kilonewtons. For H piles drive to refusal on competent bedrock the Ultimate Limit State resistance could be taken as the factored structural capacity of the pile. The tips of H-piles should be reinforced with steel plates to reduce the potential for damage.

The depth to bedrock, bedrock type and condition should be investigated if this construction option is to be pursued.

Downdrag Loads

The downdrag loads on the abutment piles due to consolidation of the grey silty clay should be considered in the pile design. The amount of downdrag loading which may occur will depend on the following factors:

- The pile type and size;
- The increase in stress due to grade raise filling and sustained loading of the approach embankments.

Estimates of the likely downdrag forces can be provided as the design progresses.

Frost Protection

The depth of frost penetration for areas which are to be cleared of snow is 1.8 metres in the Ottawa area. To prevent frost jacking of the foundations due to adfreeze within the zone of frost penetration, the foundations should be backfilled with free draining, non-frost susceptible material such as Granular B Type I within the depth frost penetration. The imported granular backfill material should be compacted in maximum 200 millimetre thick lifts to at least 95 percent of the standard Proctor dry density.

Abutment Backfill and Drainage

The abutments should be backfilled with clean, non-frost susceptible sand, such as that meeting OPSS requirements for Granular B Type I. To reduce the effects of the weight of the fill material on the grey silty clay deposit, the sand should be free of gravel and cobble size material, and should have a standard Proctor dry density of less than about 19 kilonewtons per cubic metre. The extent of the wall backfill material should be in accordance with Figure C-6.20 in the CHBDC. For design purposes, the horizontal extent of frost penetration in overburden could be taken as 1.8 metres. The sand backfill material should be placed in maximum 200 millimetre thick lifts and should be compacted to at least 95 percent of the standard Proctor dry density value.

The lateral pressure on the abutments due to the earth loads will depend on the amount of movement that occurs at the top of the abutments as a result of the earth pressures. Further details on the horizontal earth pressures could be provided if this option is to be pursued.

Approach Embankments

If the vertical roadway profile is to be raised as part of the bridge construction works settlement of the silty clay beneath the approach embankments should be considered. It is suggested that any increase in grade be kept to a minimum.

Seismic Site Coefficient

The depth of investigation of the boreholes advanced during this investigation is not considered sufficient to adequately characterize the subsurface conditions for the seismic Site Class. However, the existing geology maps of the area indicate that bedrock is present at a depth of about 10 to 15 metres. As such, according to the CHBDC, the site conforms to soil profile Type I, since the culvert will likely be founded on stable deposits of stiff clays, having a thickness less than 60 metres. Therefore, a Site Coefficient, S, of 1.0 could be used to calculate the earthquake loads on the structure.

Corrosion of Buried Concrete and Steel

The measured sulphate concentration in the creek water sample from the culvert is 57 milligrams per litre. The measured sulphate in the soil sample recovered from borehole 11-1 116 at about 1.8 metres below ground surface is 116 micrograms per gram or about 0.01 percent. According to Canadian Standards Association (CSA) “Concrete Materials and Methods of Concrete Construction”, the concentration of sulphate in the creek water and soil can be classified as low. For low exposure conditions, any concrete that will be in contact with the native soil or groundwater should be batched with General Use (formerly known as Type 10 cement). The design of any concrete should take into consideration freeze thaw effects and the presence of chlorides.

Based on the conductivity and pH of the creek water, the sample of the creek water recovered can be classified as slightly aggressive towards unprotected steel. It is noted that the corrosivity of the surface water could vary throughout the year due to the application sodium chloride for de-icing or fertilizer use in the nearby field.

ADDITIONAL CONSIDERATIONS

The details for the proposed construction were not available to us at the time of preparation of this report. It is recommended that the final design drawings be reviewed by the geotechnical engineer to ensure that the guidelines provided in this report have been interpreted as intended.

The engagement of the services of the geotechnical consultant during construction is recommended to confirm that the subsurface conditions throughout the proposed excavation do not materially differ from those given in the report and that the construction activities do not adversely affect the intent of the design. The subgrade surface should be inspected by experienced geotechnical personnel to ensure that suitable materials have been reached and properly prepared. The placing and compaction of earth fill and imported granular materials should be inspected to ensure that the materials used conform to the grading and compaction specifications.

We trust that this report is sufficient for your requirements. If you have any questions concerning this information or if we can be of further assistance to you on this project, please call.

Yours truly,

HOULE CHEVRIER ENGINEERING LTD.

Dáire Cummins, M.Sc., D.I.C.

Andrew Chevrier, M.Eng., P.Eng.
Principal

LICENSED PROFESSIONAL ENGINEERS
19 OCT 2011
A.F. CHEVRIER
7974504
PROVINCE OF ONTARIO

List of Abbreviations and Terminology
Record of Borehole sheets
Figures 1 and 2
Attachments A and B

LIST OF ABBREVIATIONS AND TERMINOLOGY

SAMPLE TYPES

AS auger sample
CS chunk sample
DO drive open
MS manual sample
RC rock core
ST slotted tube
TO thin-walled open Shelby tube
TP thin-walled piston Shelby tube
WS wash sample

PENETRATION RESISTANCE

Standard Penetration Resistance, N
The number of blows by a 63.5 kg hammer dropped 760 millimetres required to drive a 50 mm drive open sampler for a distance of 300 mm. For split spoon samples where less than 300 mm of penetration was achieved, the number of blows is reported over the sampler penetration in mm.

Dynamic Penetration Resistance
The number of blows by a 63.5 kg hammer dropped 760 mm to drive a 50 mm diameter, 60^{o} cone attached to 'A' size drill rods for a distance of 300 mm.

WH
Sampler advanced by static weight of hammer and drill rods.

WR
Sampler advanced by static weight of drill rods.

PH
Sampler advanced by hydraulic pressure from drill rig.

PM
Sampler advanced by manual pressure.

SOIL TESTS

C consolidation test
H hydrometer analysis
M sieve analysis
MH sieve and hydrometer analysis
U unconfined compression test
Q undrained triaxial test
V field vane, undisturbed and remoulded shear strength

SOIL DESCRIPTIONS

Relative Density	'N' Value
Very Loose	0 to 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very Dense	over 50

Consistency	Undrained Shear Strength (kPa)
Very soft	0 to 12
Soft	12 to 25
Firm	25 to 50
Stiff	50 to 100
Very Stiff	over 100

LIST OF COMMON SYMBOLS

c_u undrained shear strength
e void ratio
C_c compression index
c_v coefficient of consolidation
k coefficient of permeability
I_p plasticity index
n porosity
u pore pressure
w moisture content
w_L liquid limit
w_P plastic limit
ϕ^1 effective angle of friction
γ unit weight of soil
γ^1 unit weight of submerged soil
σ normal stress

RECORD OF BOREHOLE 11-1

PROJECT: 11-304

LOCATION: See Borehole Location Plan, Figure 2

BORING DATE: June 29, 2011

SHEET 1 OF 1

DATUM: Geodetic

SPT HAMMER: 63.5 kg, 0.76m drop

BORING METHOD: Portable Uncased

Description	Elev. (m)	Depth (m)
Ground Surface	41.68	
TOPSOIL FILL	41.58	0.10
Grey brown silty clay, some gravel, cobbles and boulders containing refuse (FILL MATERIAL)	39.85	1.83
Stiff, grey SILTY CLAY	35.58	6.10
End of Borehole		

Sample Number	Type	Blows/0.3m
1	50 DO	1
2	50 DO	5
3	50 DO	5
4	50 DO	3
5	50 DO	7
6	50 DO	9
7	50 DO	7

DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m: 20 40 60 80

SHEAR STRENGTH Cu, kPa: nat. V - + Q - ● rem. V - ⊕ U - ○ : 20 40 60 80

HYDRAULIC CONDUCTIVITY, k, cm/s: 10^{-7} 10^{-6} 10^{-5} 10^{-4}

WATER CONTENT, PERCENT: Wp W Wl : 20 40 60 80

ADDITIONAL LAB. TESTING

PIEZOMETER OR STANDPIPE INSTALLATION: Native Backfill

DEPTH SCALE METRES: 0 1 2 3 4 5 6 7 8

DEPTH SCALE 1 to 40

Houle Chevrier Engineering Ltd.

LOGGED: A.N.

CHECKED:

BOREHOLE RECORD 2011 11-304.GPJ HCE DATA TEMPLATE.GDT 9/21/11

RECORD OF BOREHOLE 11-2

PROJECT: 11-304

LOCATION: See Borehole Location Plan, Figure 2

BORING DATE: June 29, 2011

SHEET 1 OF 1

DATUM: Geodetic

SPT HAMMER: 63.5 kg, 0.76m drop

BORING METHOD: Portable Uncased

SOIL PROFILE – DESCRIPTION	ELEV. / DEPTH (m)	SAMPLE NUMBER	TYPE	BLOWS/0.3m
Ground Surface	42.20			
TOPSOIL FILL	42.05 / 0.15			
Very stiff, grey brown silty clay, some organic material (FILL MATERIAL)		1	50 DO	6
		2	50 DO	2
Stiff to firm, grey silty clay (FILL MATERIAL)	40.90 / 1.30	3	50 DO	2
Stiff to firm, grey silty clay, some organic material, trace gravel (Possible former TOPSOIL)	39.48 / 2.72	4	50 DO	5
Stiff to very stiff, grey SILTY CLAY, with black mottling	39.33 / 2.87			
		5	50 DO	8
		6	50 DO	5
		7	50 DO	7
		8	50 DO	10
		9	50 DO	7
End of Borehole	34.88 / 7.32			

DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m: 20 40 60 80

SHEAR STRENGTH Cu, kPa: nat. V - + Q - ● rem. V - ⊕ U - ○ : 20 40 60 80

HYDRAULIC CONDUCTIVITY, k, cm/s: 10^{-7} 10^{-6} 10^{-5} 10^{-4}

WATER CONTENT, PERCENT: Wp W Wl: 20 40 60 80

ADDITIONAL LAB. TESTING

PIEZOMETER OR STANDPIPE INSTALLATION: Native Backfill

DEPTH SCALE METRES: 0 1 2 3 4 5 6 7 8

>>

DEPTH SCALE 1 to 40

Houle Chevrier Engineering Ltd.

LOGGED: A.N.

CHECKED:

KEY PLAN

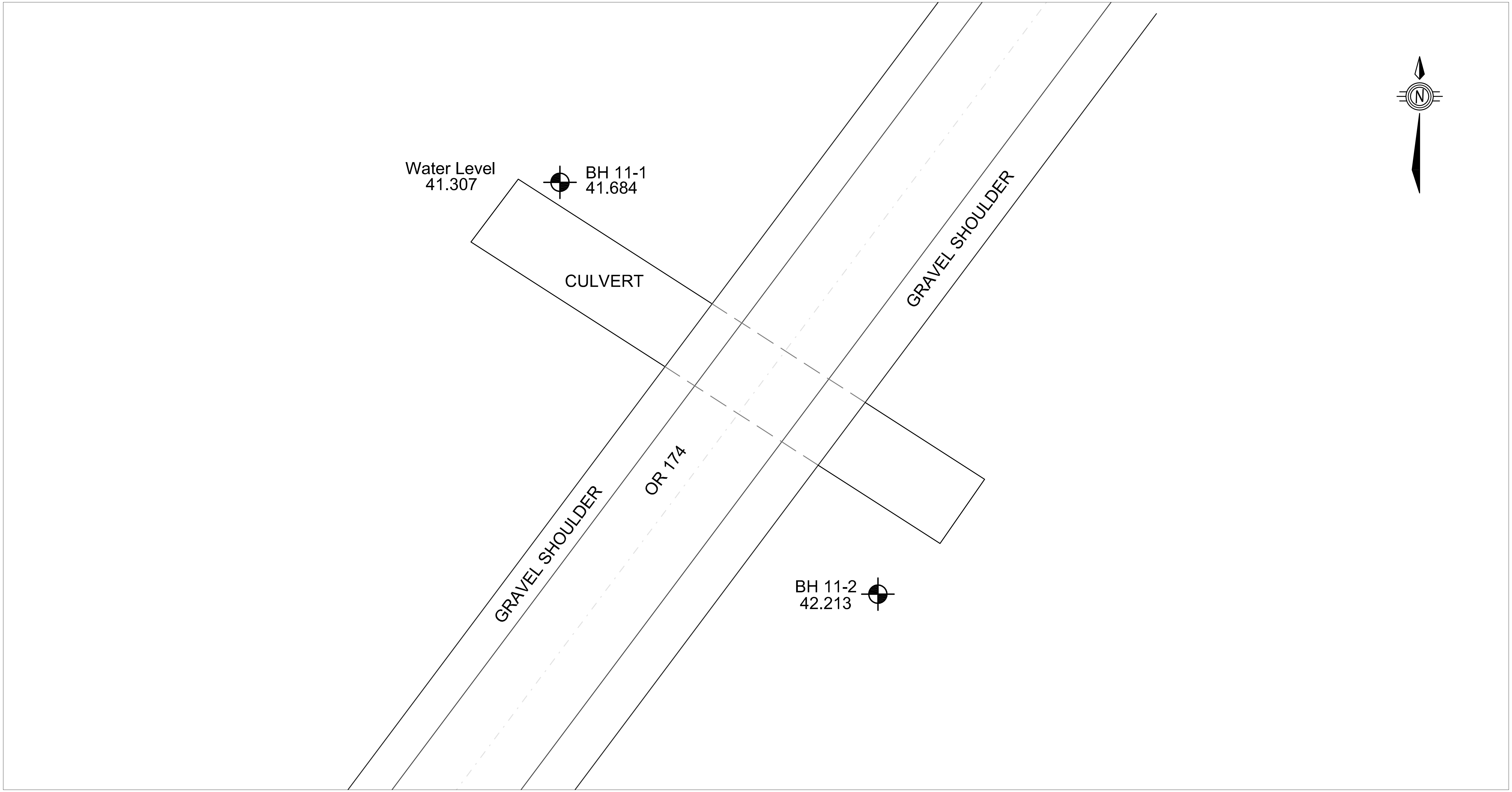
FIGURE 1

N.T.S

Houle Chevrier Engineering

Date: October 2011

Project: 11-304-1



LEGEND

BH 11-1 APPROXIMATE BOREHOLE LOCATION IN PLAN, CURRENT INVESTIGATION BY HOULE CHEVRIER ENGINEERING LTD.

Client	GENIVAR	Location	EAST CULVERT HIGHWAY 174	Revision 0
Drawn by A.N.	Approved by D.C.	Project No. 11-304-1		Scale 1:300
Houle Chevrier Engineering		Title	BOREHOLE LOCATION PLAN	
		Date	October 2011	FIGURE 2

ATTACHMENT A

RESULTS OF CREEK WATER SAMPLE

REPORT OF ANALYSIS

Client: **Houle Chevrier Engineering**
180 Wescar Lane, R.R. #2

Carp, ON
K0A 1L0
Attention: **Daire Cummins**

Report Number: 1116485
Date: 2011-07-26
Date Submitted: 2011-07-21

Project: 11-304

P.O. Number:
Matrix: Water

Chain of Custody Number: **130016**

								GUIDELINE		
		LAB ID:	897595							
		Sample Date:	2011-07-21					ODWSOG		
		Sample ID:	Cardinal Creek							
PARAMETER	**UNITS**	**MRL**						**TYPE**	**LIMIT**	**UNITS**
Chloride	mg/L	1	67					AO	250	mg/L
Conductivity	uS/cm	5	715							
pH			8.24						6.5-8.5	
Sulphate	mg/L	1	57					AO	500	mg/L
Hardness as CaCO3	mg/L	1	211					OG	100	mg/L
Calcium	mg/L	1	50							
Magnesium	mg/L	1	21							

MRL = Method Reporting Limit INC = Incomplete AO = Aesthetic Objective OG = Operational Guideline MAC = Maximum Allowable Concentration IMAC = Interim Maximum Allowable Concentration

Comment:

APPROVAL: ____________________
Ewan McRobbie
Inorganic Lab Supervisor

Methods references and/or additional QA/QC information available on request.

ATTACHMENT B

RESULTS OF SOIL SAMPLE

PARACEL LABORATORIES LTD.
TRUSTED. RESPONSIVE. RELIABLE.
300-2319 St. Laurent Blvd.
Ottawa, Ontario K1G 4J8
p: 1-800-749-1947
e: paracel@paracellabs.com
www.paracellabs.com
OTTAWA • NIAGARA FALLS • MISSISSAUGA • SARNIA

Certificate of Analysis

Houle Chevrier
180 Wescar Lane
Carp, ON K0A 1L0
Attn: Daire Cummins

Phone: 613-836-1422
Fax: (613) 836-9731

Client PO:
Project: 11-304
Custody: 70436

Report Date: 13-Jul-2011
Order Date: 7-Jul-2011
Order #: 1128228

This Certificate of Analysis contains analytical data applicable to the following samples as submitted:

Paracel ID	Client ID
1128228-01	BH201 SA3
1128228-02	BH103 SA4

Approved By: Mark Foto

Mark Foto, M.Sc. For Dale Robertson, BSc
Laboratory Director

PARACEL LABORATORIES LTD.

Order #: 1128228

Certificate of Analysis

Report Date: 13-Jul-2011

Client: **Houle Chevrier**

Order Date:7-Jul-2011

Client PO:

Project Description: 11-304

Analysis Summary Table

Analysis	Method Reference/Description	Extraction Date	Analysis Date
Anions	EPA 300.1 - IC, water extraction	13-Jul-11	13-Jul-11
pH	EPA 150.1 - pH probe, CaCl buffered extraction	11-Jul-11	12-Jul-11
Resistivity	EPA 120.1 - probe, water extraction	12-Jul-11	13-Jul-11
Solids, %	Gravimetric, calculation	13-Jul-11	13-Jul-11

PARACEL LABORATORIES LTD.

Order #: 1128228

Certificate of Analysis

Report Date: 13-Jul-2011

Client: **Houle Chevrier**

Order Date:7-Jul-2011

Client PO:

Project Description: 11-304

	Client ID:	BH201 SA3	BH103 SA4	-	-
	Sample Date:	29-Jun-11	30-Jun-11	-	-
	Sample ID:	1128228-01	1128228-02	-	-
	MDL/Units	Soil	Soil	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	67.6	72.6	-	-
General Inorganics					
pH	0.1 pH Units	7.1	6.9	-	-
Resistivity	0.10 Ohm.m	8.70	28.4	-	-
Anions					
Chloride	5 ug/g dry	812	177	-	-
Sulphate	5 ug/g dry	116	56	-	-

Method Quality Control: Blank

Analyte	Result	Reporting Limit	Units	Source Result	%REC	%REC Limit	RPD	RPD Limit	Notes
Anions									
Chloride	ND	5	ug/g						
Sulphate	ND	5	ug/g						
General Inorganics									
Resistivity	ND	0.10	Ohm.m						

Method Quality Control: Duplicate

Analyte	Result	Reporting Limit	Units	Source Result	%REC	%REC Limit	RPD	RPD Limit	Notes
Anions									
Chloride	ND	5	ug/g dry	ND				20	
Sulphate	ND	5	ug/g dry	ND				20	
General Inorganics									
pH	8.0	0.1	pH Units	8.0			0.6	10	
Resistivity	8.62	0.10	Ohm.m	8.70			0.9	20	
Physical Characteristics									
% Solids	92.8	0.1	% by Wt.	88.9			4.3	25	

Method Quality Control: Spike

Analyte	Result	Reporting Limit	Units	Source Result	%REC	%REC Limit	RPD	RPD Limit	Notes
Anions									
Chloride	86.8	5	ug/g	ND	86.8	78-113			
Sulphate	97.7	5	ug/g	ND	97.7	78-111			

Sample and QC Qualifiers Notes

None

Sample Data Revisions

None

Work Order Revisions/Comments:

None

Other Report Notes:

n/a: not applicable
MDL: Method Detection Limit
Source Result: Data used as source for matrix and duplicate samples
%REC: Percent recovery.
RPD: Relative percent difference.
Soil results are reported on a dry weight basis when the units are denoted with 'dry'.
Where %Solids is reported, moisture loss includes the loss of volatile hydrocarbons.