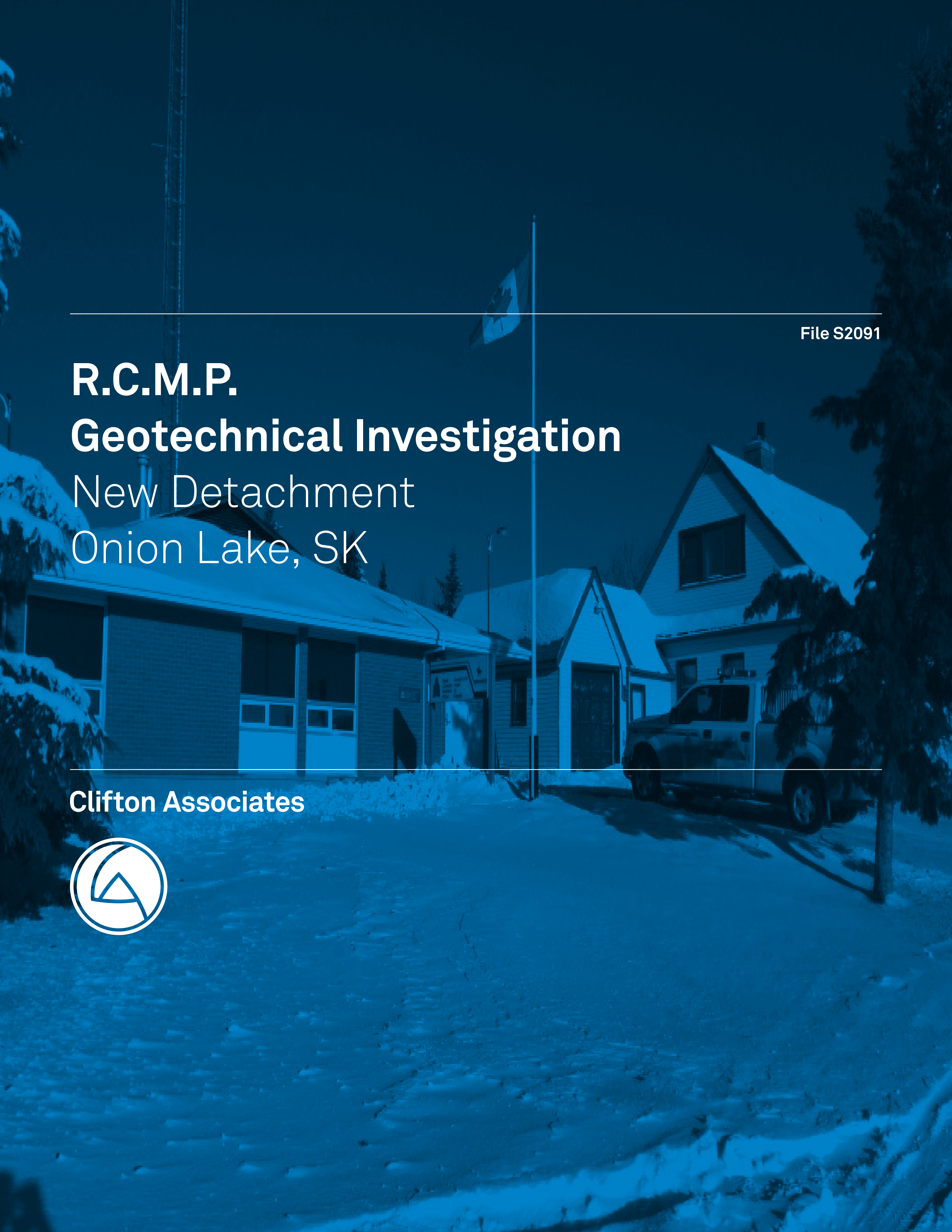

File S2091

R.C.M.P.
Geotechnical Investigation

New Detachment
Onion Lake, SK

Clifton Associates

Clifton Associates

11 March 2015

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R.C.M.P.
New Detachment
Onion Lake, SK

File S2091

We are pleased to present to you our geotechnical report regarding the above subject.

Should you have any questions or concerns regarding this report, please contact me.

Yours truly,

Clifton Associates Ltd.

Richard T Yoshida PEng
Senior Geotechnical Engineer
RTY/djb

Distribution: R.C.M.P. - 3 copies
Clifton Associates Ltd. - 2 copies

Table of Contents

Appendices

List of Tables

1.0 Introduction

This report presents results of a geotechnical investigation conducted for the new RCMP Detachment Facility to be constructed in Onion Lake, Saskatchewan. The Site is located on part of SW5-55-27-W3 in Onion Lake, Saskatchewan as shown on Drawing No. S2091-01. Authorization to proceed was received by RCMP Purchase Order No. 7201596 dated 29 December 2014 via email from Bonny Manz, Senior Contracting Officer, Procurement & Contracting Services, RCMP.

In general, the objectives of this work were:

- To define the subsurface soil strata and groundwater conditions in the area of the proposed development.
- To provide recommendations for suitable methods of foundation support for proposed structures.
- To provide pavement structure recommendations.
- To provide recommendations for excavations, backfill and drainage.
- To provide general site development criteria.
- To provide commentary on pertinent geotechnical issues identified during the subsurface investigation.

2.0 Description of the Site and Proposed Structures

The new structure will be constructed at the existing detachment. It is our understanding that the new detachment facility will consist of a one storey structure with a footprint of about 765 m^2, and a separate 75 m^2 storage building. It is anticipated that the detachment building may have a mezzanine or crawl space, or both, with a footprint smaller than the main floor.

The current detachment property is bordered by undeveloped grassy areas on the north and east, with bluffs of trees along the property edge. Access roads border the west and south, where there are various residences.

A buried telephone line runs east from the RCMP residence around to the south of the detachment. Buried power runs between the garage and holding cells to the shed, then follows the driveway to the north and south. Natural gas follows the driveway towards the BH101 area and continues northwest towards the holding cells and residence. Underground sewer lines exist on the Site but were not marked. Overhead power entered the Site and ran to the detachment from across the entrance road.

General foundation recommendations contained herein are provided for the proposed structures. These recommendations can be revised for specific loadings or configurations, if required, once additional details are known. This office must be advised of any changes so that the applicability of these recommendations can be assessed.

3.0 Field and Laboratory Investigation

Subsurface conditions were investigated by three boreholes drilled at the site as shown on Drawing No. S2091-02. Boreholes were drilled on 15 January 2015 using a truck mounted MARL M10 drill rig using 150 mm diameter solid stem auger. The boreholes were drilled to a maximum depth of 12.2 m below surface. Bluffs of trees and utilities prevented the drilling of additional boreholes.

Representative disturbed and undisturbed samples were recovered for laboratory analysis. Sampling was started at a depth of 0.75 m and continued at a 0.75 m interval to 4.6 m. After this depth, the sampling interval was increased to 1.5 m. Thin wall tube (Shelby) samples were collected in two boreholes, with disturbed cutting samples collected in the remaining borehole.

A standpipe piezometer was installed in Borehole BH102 to monitor groundwater levels. The piezometer was constructed using 50 mm diameter Schedule 40 PVC pipe with a machined screen. Filter sand was placed around the screen to an elevation of approximately 0.3 m above, and the remainder of the annulus was filled with bentonite chips.

Borehole locations were recorded using a handheld GPS unit. The accuracy of the measurements is not known. Borehole elevations were not measured.

The natural water content of each sample was determined. Other testing included determination of Atterberg limits, water soluble sulphate content and maximum dry density of selected representative samples. The undrained shear strength of soils was measured using a pocket penetrometer and laboratory vane shear apparatus.

Observations made during the field investigation, visual descriptions and the results of laboratory tests are recorded in the Borehole Logs, and the Summary of Sampling and Laboratory Test Data which are appended to this report. An explanation of the symbols and terms used in the borehole logs is included in the Symbols and Terms section of this report.

Laboratory testing was conducted in accordance with procedures and methodologies described in ASTM standards. The determination of the Unified Soil Classification (USC) in accordance with ASTM D2487 includes the measurement of grain size distribution with respect to gravel, sand and silt and clay sized particles. It also includes the laboratory measurement of plasticity, including plastic limit and liquid limit in accordance with ASTM D4318. Atterberg limits and determination of the plasticity of soil provides more useful information on the effect of the clay sized fraction on soil behaviour. The Standard Proctor test (ASTMD698) was used to determine the maximum dry density and optimum water content of soil.

4.0 Analysis

4.1 Stratigraphy

Stratigraphy generally consisted of glacial till to the depth of exploration, with the exception of Borehole BH102, where clay with sand was encountered to a depth of about 7.6 m. The thickness of organic topsoil varied from about 75 mm to 150 mm. Frozen soil was encountered within the upper 1 m.

Glacial till was encountered in Boreholes BH101 and BH103 to a depth of 7.6 and 9.1 m, respectively. Till was comprised predominantly of a sandy clay matrix with some silt and a trace of gravel. It was moist, and varied in consistency from very stiff to hard, with an estimated undrained shear strength of about 100 kPa to 285 kPa or more. Till generally possessed low plasticity, although till encountered in Borehole BH101 likely possessed medium to high plasticity. A sand layer about 0.6 m in depth was encountered in Borehole BH101 at a depth of 3 m.

Clay and sand encountered in Borehole BH102 was moist, and very stiff in consistency, with an estimated undrained shear strength of 150 kPa. Clay possessed high plasticity.

Unoxidized glacial till was encountered between a depth of 7.6 m and 10.7 m. It had a similar texture to the oxidized till, was moist, and very hard in consistency, with an estimated undrained shear strength of 285 kPa or more.

Standard Proctor tests were conducted to determine the maximum dry density and optimum water content of the upper till and clay. Composite samples were collected in Boreholes BH101 and BH102 in the upper 1 m. The maximum dry densities and optimum water contents for till and clay were 1,645 kg/m^3 at 18.8% and 1,600 kg/m^3 at 20.8%, respectively.

Cobbles and boulders are common within glacial till and were encountered during drilling, and should be expected in excavations and during piling.

4.2 Groundwater Regime

A standpipe piezometer was installed in Borehole BH102 with the tip placed at a depth of 12.2 m. Groundwater levels were measured on 30 January 2015, approximately two weeks after installation. The groundwater level was 2.2 m below ground surface. Although significant seepage was not observed during drilling, seepage should be expected in excavations below a depth of about 2 m due to the relatively high groundwater level, and from randomly occurring sand and sandy lenses or strata within the till strata.

Groundwater levels are expected to fluctuate with the level of development in the area, as well as seasonal changes in precipitation, infiltration and evaporation. It is not possible to predict increases in groundwater levels with precision; however, it is not unusual for groundwater levels to increase over time after development. Groundwater levels may rise temporarily due to irrigation or snowmelt as water infiltrates the surface and flows vertically and horizontally through fractures and fissures in the upper clay and till. This water may report to excavations or crawlspaces with time.

5.0 General Discussion

The major geotechnical issues associated with this project are:

- Seismic site characterization and design parameters.
- Frost susceptibility and heave.
- Foundations to support the proposed structures.
- Site development criteria, including stable cut and fill slopes.
- Roadway surfacing.
- Excavations.

5.1 Seismic Site Response

The site classification for seismic site response, as described in NBCC 2010 (Table 4.1.8.4A), can be based on the average estimated undrained shear strength or average standard penetration test blow count in the upper 30 m. The undrained shear strength of soil encountered at this site was generally greater than 100 kPa. On this basis, design can assume Site Class C conditions for seismic response.

For Site Class C conditions, the acceleration based site coefficient, F_a = 1.0 and the velocity based site coefficient, F_v = 1.0. The peak ground acceleration (PGA) and the 5% damped spectral response acceleration values for 0.2, 0.5, 1.0, and 2.0 second periods, $S_a(T)$, for the site are summarized in Table 5.1.

Table 5.1 Seismic Data

$S_a(0.2)$	$S_a(0.5)$	$S_a(1.0)$	$S_a(2.0)$	PGA
0.055	0.034	0.016	0.005	0.019

5.2 Frost

Clay with sand will have low to moderate frost susceptibility. The depth of freezing in this area will vary, depending on air temperature, ground cover, the type of any fill material utilized during development, and other factors. Frost heave can be an issue for unheated structures. The depth to groundwater is dependent upon the ground elevation. Groundwater levels were determined to be about 2.2 m. In general, frost heave may be a potential issue for foundations constructed less than about 2.5 m from surface, which is the estimated depth of freezing. Since the groundwater level is higher than the maximum expected depth of freezing, increased heave due to ice segregation could occur. At this Site, the risk can be minimized by constructing footings below the depth of freezing, supporting structures on deep foundations.

The depth of the foundation can be reduced if the foundation is insulated. This will only apply to a structure without a crawlspace or a heated crawlspace. Insulation can be incorporated into an unheated structure, although the amount of insulation required can be substantial.

The depth of burial for water lines or other lines that cannot be allowed to freeze should consider local practice. In general, it is recommended that water lines be buried at least 2.5 m below ground surface or finished grade to reduce the risk of freezing. Shallower lines can be protected using heat trace or closed cell extruded polystyrene insulation. The amount and extent of insulation required will be dependent on several factors, particularly the thermal regime around the pipe, including the depth of burial, surface conditions and fluid temperature, if present.

5.3 Foundation Alternatives

The selection of a suitable foundation will depend on the magnitude of loading and the required performance. Foundation alternatives at this site include:

- Spread footings.
- Cast in place concrete piles.
- Driven steel or timber piles.
- Helical piles.

Issues related to foundation construction will include:

- Potential for seepage or sloughing, or both, from sand layers.
- Cobbles and boulders within the till stratum.
- Frost penetration.

Shallow spread footings constructed on high plasticity clay soil within the upper metre or so will be subject to significant vertical movement, estimated to be 100 mm to 150 mm or more, associated with changes in soil moisture that can occur seasonally and over time. Vertical movement will be reduced if the spread footing is constructed below the expected depth of freezing where seasonal changes in soil moisture are less pronounced. Vertical movement will also be reduced if spread footings are constructed on low plasticity clay till.

Spread footings are not recommended unless some total and differential vertical movement associated with heave is acceptable. Foundations should be constructed below the anticipated depth of freezing of 2.5 m to minimize the risk of frost heave. Foundations may incorporate insulation to limit heat loss and to ensure that soil under that foundation does not freeze.

Potential heave associated with freezing of clay is estimated to be about 45 mm per metre depth of soil below the foundation and to the estimated depth of freezing, which is based on the approximate void ratio of clay and the volumetric increase as water freezes. Heave may be greater if ice segregation occurs.

Augered cast in place concrete piles are a suitable alternative for this site. Some difficulty can be expected during piling due to cobbles or boulders within the till stratum. Temporary sleeving will likely be required due to potential sloughing from sand or sandy layers, as well as seepage from fractures or fissures within the till.

Driven steel or timber piles are a foundation alternative, although moderately difficult driving conditions are anticipated due to the random presence of cobbles or boulders. Driven piles may be designed on the basis of skin friction and end bearing. It is possible that driven piles will refuse on cobbles or boulders, if encountered. Steel pipe piles may be driven open ended and filled with concrete after they have been driven to final depth. A soil plug will form while driving, which will increase the effective end bearing area to the gross cross section of the pipe. The box area of an H section pile, defined as the width times the depth, may be used to calculate the end bearing of an H section steel pile.

Helical piles developing their capacity on the basis of skin friction and end bearing may be a suitable alternative. Skin friction along the pile shaft is generally considered to contribute to capacity for a shaft diameter greater than 100 mm. The lateral load carrying capacity of helical piles should be verified as part of the structural design where significant moment associated with lateral loading is present.

5.4 Coefficient of Earth Pressure

Active and passive earth pressure can be calculated using active earth pressure coefficients. Table 5.2 provides a summary of these properties.

Table 5.2 Earth Pressure Coefficients

Material	Angle of Internal Friction	Total Unit Weight	Earth Pressure Coefficients		
	(°)	(kN/m^3)	Active	At-Rest	Passive
Clay	19	18.0	0.51	0.67	2.0
Glacial till	33	19.0	0.29	0.46	3.4
Granular fill	38	20.0	0.24	0.38	4.2

5.5 Coefficient of Friction for Sliding

The friction angle between concrete and soil for concrete poured directly on soil can be assumed to be equal to the angle of internal friction for soil provided in Table 5.4. This assumes a rough contact surface between soil and concrete. For smooth concrete against soil, the tangent of the angle of internal friction should be reduced by 20 percent.

5.6 Modulus of Subgrade Reaction

The modulus of subgrade reaction, k_s (MPa), was estimated on the basis of correlation with undrained shear strength. The modulus of subgrade reaction is nonlinear and depends on the soil deformation. The modulus of subgrade reaction can be estimated as:

$$k_s = \frac{250}{t}$$

where t = estimated settlement (mm) corresponding to the applied load. Thus, for an estimated settlement of 25 mm, the modulus of subgrade reaction, k_s = 10 MPa.

The value for the modulus of subgrade reaction should be varied over a range of about ±50% to assess the sensitivity of performance to the assumed value.

5.7 Permeability

The permeability or hydraulic conductivity of in situ clay with sand or glacial till has been estimated to be about 10^{-6} m/s to 10^{-7} m/s, based on typical values for an oxidized till that is fractured and fissured. The hydraulic conductivity of soil that is scarified, moisture conditioned, and recompacted will be significantly lower when secondary structure is destroyed.

5.8 Potential for Liquefaction

Factors influencing liquefaction include soil type, relative density, confining pressure, soil drainage conditions, and seismic conditions. Sand can be susceptible to liquefaction if it is loose and has poor drainage, and if ground accelerations associated with an earthquake or other event is sufficient. Soil in this area will not be susceptible to liquefaction.

5.9 Floors

Significant vertical movement, estimated to be 100 mm to 150 mm or more, associated with changes in soil moisture can be expected for floors constructed on high plasticity clay encountered in the vicinity of Boreholes BH101 and BH102. This will generally result in some cracking of the floor and loss of utility. Performance of floors will be improved if they are constructed on low plasticity glacial till encountered in the vicinity of Borehole BH103.

The amount of potential heave can be reduced if the clay subgrade is excavated and replaced with a clayey soil with low plasticity, or with a granular soil. The magnitude of the reduction in potential heave will be dependent on the depth to which clay is replaced. For example, replacing about 900 mm of clay will reduce the potential heave by about 75% so that the potential heave will be less than about 25 mm to 40 mm. Replacing 1,200 mm of clay will reduce the potential heave by about 85%, corresponding to a potential heave of about 15 mm to 25 mm. If this level of performance is not acceptable, floors can be structurally supported.

Construction of grade supported floor slabs should avoid fill material of unknown composition and condition. If significant fill is contemplated, adequate compaction control and material selection criteria will be crucial to ensure suitable performance. High plasticity clay compacted dry of optimum will have a high potential for heave. High plasticity clay should be compacted at or above optimum to reduce the potential for heave. Commentary regarding compaction is provided in a subsequent section.

Any organic or soft material should be removed and the subgrade should be proof rolled to determine the location of any soft areas. These areas should be excavated and filled with compacted, well graded pit run gravel or other granular fill. The subgrade should be compacted to at least 98% of the maximum dry density as determined in accordance with the standard Proctor test.

A minimum 150 mm of compacted, crushed base course material should be placed under any floor slab. The thickness of the base course material can be increased as required to provide adequate support for the applied loading. A layer of polyethylene sheeting 150 µm (minimum) thick should be placed between the granular base and the concrete slab to deter the migration of moisture through the floor and loss of moisture from freshly placed concrete.

5.10 Excavations

Seepage and potential dewatering is expected for excavations below a depth of 2.2 m. The amount and rate of seepage may change seasonally or as a result of precipitation and infiltration.

Soil in this area will be a type 3 soil as defined by Occupational Health and Safety regulations. In general, excavations should be no steeper than about 1 horizontal to 1 vertical (1:1). Although excavations through these materials may stand in the short term at steeper angles, over steepened slopes will slough and collapse if they are left open for long periods of time or if water is allowed to infiltrate. Excavation conditions must be carefully monitored, and slopes flattened during construction if conditions warrant. Failure may be sudden and may endanger personnel and equipment working in the vicinity.

5.11 Roadway Surfacing Structures

The subgrade soil available at this Site is either clay or clay till. The estimated CBR values for clay and till will be 2.5 and 5.0. it is preferable to utilized clay till as a road subgrade to reduce the cost of the pavement structure. Recommendations for roadway structures for both a clay and glacial till subgrade have been provided. A design period of 20 years was used for all roadways and parking lots. We have made some assumptions regarding traffic.

Pavement structures are designed on the number and type of heavy trucks that are expected. Car and small truck traffic is almost insignificant, as one heavy semi-trailer truck is equivalent to perhaps 2,000 to 3,000 cars.

If traffic on the lot can be segregated into light and heavy traffic areas, there may be some economy in providing two different pavement structures. The actual traffic make-up is not known. For purposes of design, traffic has been classified as described below, assuming traffic in terms of a number of trucks on a daily or monthly basis, which can be:

- Heavy Pavements: The design is based on approximately 1 x 10^5 ESALs, which is equivalent to about two trucks per day. As a comparison, if we assume six trucks per month, the design traffic would be about 1 x 10^4 ESALs for a 20 year design period.
- Light Pavements: This includes any areas that are intended for a paved surface, but are not subject to any planned heavy truck haul. An example would be a parking lot. The structure here is nominal and design is based on the assumption that these areas could experience one loaded truck per week that may pass through or temporarily stop in these areas. The design traffic for these areas is 1 x 10^4 ESALs for a 20 year design period.

The recommended pavement structure thicknesses are presented in Table 5.3. They are based on a subgrade CBR of 2.5 for clay and 5.0 for glacial till, and the design loadings described above.

A granular pavement structure can utilize the base course thickness shown in Table 5.3. The subbase thickness should be increased to 160 mm and 260 mm for light and heavy traffic areas, respectively for till, and 310 mm and 420 mm for light and heavy traffic areas, respectively for clay.

Alternately, the surface may consist of a 100 mm to 150 mm thickness of traffic gravel placed on the subbase material. This type of structure will require regular and periodic maintenance, including blading and reapplication of a gravel surface.

A concrete slab may be considered for areas subject to heavy wheel loading and shearing stresses associated with turning of tires. The slab should be 200 mm in thickness and can be placed on a 25 mm to 50 mm thick levelling course of sand.

Table 5.3 Recommended Pavement Structure Thickness

	Thickness (mm)			
	Clay, CBR = 2.5		Clay Till, CBR = 5.0	
	Light Pavement Structures	Heavy Pavement Structures	Light Pavement Structures	Heavy Pavement Structures
Subbase	250	330	110	190
Crushed Base Course (Type 33)	150	150	150	150
Hot Mix Asphalt Concrete	50	70	40	60

5.12 General Site Development

5.12.1 Construction Equipment

Standard excavation equipment may be used for site development; no unusual excavation conditions are anticipated. Large vibratory smooth steel drum compacting equipment should be used to compact sand and granular soil. A padfoot compactor will be better suited for clay or clayey soil.

5.12.2 Topsoil, Cobbles, and Boulders

The thickness of topsoil varied from about 75 mm to 150 mm. Organic topsoil that is present should be removed prior to placement of any fill to minimize the potential for settlement.

Cobbles were occasionally encountered during drilling in till. Cobbles and boulders can be expected in excavations.

5.12.3 Groundwater

Groundwater seepage should be expected in excavations below a depth of 2.2 m. Groundwater levels are expected to fluctuate seasonally and with precipitation.

5.12.4 Suitability of On-Site Soil for Compacted Fill

Clay or glacial till at this Site should be an acceptable material for construction of embankments or fills. These materials are present at moisture contents that are close to the plastic limit; therefore, they should be readily compactable. Some moisture conditioning or drying may be necessary to aid in compaction.

5.12.5 Shrinkage Factors

For estimates of earthwork volumes, a shrinkage factor of 15% to 20% may be used for clay or glacial till.

5.12.6 Engineered Fill

If required, engineered fills supporting important structures should utilize local sand or pit run gravel. Specifications for pit run gravel and sand, and crushed base course material, are appended to this report.

5.12.7 Cut or Fill Slopes

Cut or fill slopes in clay or till will possess long term stability at slopes of 2 horizontal to 1 vertical (2:1), but may be subject to increased rates of erosion. Flatter slopes are preferred for landscaping purposes. Vegetation can be used to maintain slopes. Where vegetation is not desirable, a gravel surface with a minimum thickness of 150 mm is recommended on these slopes to reduce the potential for erosion.

Drainage swales and ditches should be constructed with gentle slopes, if possible, as the soil will be easily eroded, particularly if water velocities are greater than 2 m/s.

It is desirable to have road subgrades at least 1.0 m above natural ground on fill sections or to have at least a 1.0 m ditch in cut-fill sections. The surface of the subgrade should have enough cross-slope to ensure positive surface drainage prior to surfacing, nominally 5%.

5.12.8 Site Grading

The Site should be graded to ensure positive drainage throughout the construction phase. Grades should be created to direct water away from excavations and trenches. Within excavations, the subgrade should be graded with a cross slope so that any accumulated water can be removed by pumping.

Proper site grading design is critical to ensure good long term performance of shallow footings. Grades should ensure that water from precipitation or snowmelt does not accumulate near structures. A positive slope away from structures of at least 5% for about 3 m is recommended. Infiltration rate into the sand is expected to be high.

5.13 Compaction Specifications

Compaction specifications must consider the desired properties of the fill. Specifications will typically require compaction to a percentage of the maximum dry density, determined in accordance with the standard Proctor test, and may include a range of water contents that are desirable. Depending on the desired properties for the compacted soil, the water content is often provided as a guide to the contractor, since the compactive effort will usually be minimized if the soil is compacted close to the optimum water content determined in accordance with the standard Proctor test. If the soil is wet of optimum, it will be possible to attain a specified density if greater compactive effort or more work is applied to the soil.

The compaction water content will have an impact on the properties of the compacted soil. Soil strength and compressibility is better if the water content is lower than optimum. Soil compacted wet of optimum to the necessary density may be more compressible under low pressure and may have reduced strength. The potential for heave will be increased if high plasticity clay is compacted dry of optimum.

The following recommendations are provided for compaction:

- The excavated subgrade should be uniformly compacted to 95% of its maximum dry density, determined in accordance with ASTM D698-00a, Standard Test Methods for Laboratory Compaction Characteristics of Soil Using Standard Effort [12,400 ft-lbf/ft^3 (600 kN-m/m^3)]. The water content of the subgrade should be close to optimum water content.
- Soft areas in the subgrade should be subcut and backfilled with local sand or well graded pit run gravel that is uniformly compacted to at least 100% of its maximum dry density, determined in accordance with ASTM D698-00a, Standard Test Methods for Laboratory Compaction Characteristics of Soil Using Standard Effort [12,400 ft-lbf/ft^3 (600 kN-m/m^3)].
- If considered, granular pads for shallow spread footings should be constructed with local sand or a well graded pit run gravel that conforms to the recommended gradations for granular materials appended to this report. The material should be compacted to a minimum average 98% of maximum dry density for four (4) consecutive tests, with no single test less than 96%, determined in accordance with the standard Proctor test. Lift thickness should not exceed 200 mm.
- Crushed base course that will be under a floor slab, spread footing, or paved area should be compacted to a minimum 98% of its maximum dry density, determined in accordance with the standard Proctor test.
- Fill material that will be under a parking area or roadway should be compacted to a minimum 98% of maximum dry density, determined in accordance with the standard Proctor test, in lifts no thicker than 150 mm in compacted thickness. Fill under landscaped areas does not generally require high density, although some compaction is required to reduce the amount of

settlement. A suggested level of compaction is a minimum 90% of maximum dry density, determined in accordance with the standard Proctor test.

- Backfill of trenches in areas that already have been compacted should be with new subbase material as specified previously, and compacted to a minimum 98% of maximum dry density, determined in accordance with the standard Proctor test.
- Backfill and compact simultaneously each side of walls in layers of 300 mm to ensure that excessive pressure is not applied to one side of the wall.

5.14 Potential for Sulphate Attack

Water soluble sulphate contents were measured as high as 3.56% by dry weight of soil in the upper few metres. According to CSA A23.1, the potential for sulphate attack is very severe, corresponding to an S-1 class of exposure. On this basis, sulphate resistant cement (Type HS) must be specified for all concrete in contact with the native soil. The maximum water to cement ratio should be 0.40, with a minimum specified compressive strength of 35 MPa at 56 days. Additional recommendations regarding sulphate resistant cement may be found in CSA A23.1.

5.15 Soil pH

Sample numbers JW02, JW14 and JW26 from Boreholes BH101, BH102 and BH103, respectively, were submitted to ALS Laboratory Group in Saskatoon for determination of pH. The measured pH ranged from 7.33 to 7.66. The laboratory Certificate of Analysis for the pH testing is included in Appendix D.

6.0 Discussion of Foundations

Foundation alternatives at this Site include shallow spread footings constructed on clay or glacial till, cast in place concrete piles, driven steel or timber piles, or helical piles. Some construction difficulties should be expected for deep foundations due to the presence of cobbles and boulders.

6.1 Shallow Spread Footings

Generally, spread footings must be constructed below the anticipated depth of freezing, which in this area could be about 2.5 m. There is a risk of frost heave associated with ice segregation for footings constructed above this depth since groundwater will be close to the freezing front. Spread footings constructed on high plasticity encountered in some areas of the site will be subject to significant total and differential vertical movement that can result in some cracking. If this performance is not acceptable, a deep foundation should be considered.

6.1.1 Design

Clay and till at this Site possesses adequate bearing capacity for lightly loaded structures utilizing shallow footings for foundation support. The estimated net ultimate bearing capacity of clay or till in the upper 5 m is 400 kPa. Resistance factors for shallow foundations from NBCC 2010 have been summarized in Table 6.1.

Table 6.1 Resistance Factors for Shallow and Deep Foundations

Case	Resistance Factor
Shallow Foundations: Vertical resistance by semi-empirical analysis using laboratory and in situ test data	0.5
Deep Foundation: Bearing resistance to axial load based on semi-empirical analysis using laboratory and in situ test data	0.4
Deep Foundation: Analysis using dynamic monitoring results	0.5
Deep Foundation: Analysis using static loading test results	0.6
Uplift Analysis: By semi-empirical analysis	0.3
Uplift Analysis: Using load test results	0.4
Horizontal Load Resistance	0.5

For a spread footing, the geotechnical resistance calculated using the ultimate bearing capacity and appropriate resistance factor is utilized to ensure that a gross failure of the foundation does not occur. Settlement considerations will typically govern the selection of an appropriate bearing pressure. Consideration of serviceability utilizes working or services loads and unfactored geotechnical properties for soil strata. The estimated serviceability limit pressure for a spread footing is 200 kPa.

6.1.2 Settlement

The amount of settlement will be dependent on factors such as the foundation size and applied pressure. The amount of settlement will increase for a larger foundation with no change in the bearing pressure. The estimated settlement for a spread footing with a width of about 450 mm will be less than about 25 mm.

6.1.3 Subgrade Preparation

The width of excavations for major foundations should extend a minimum 1.5 m plus the depth of excavation beyond the edge of the foundation to ensure that the subgrade can be prepared and fill can be properly placed and compacted.

Although the base of large excavations can be level, it is desirable to create a cross slope on the subgrade to encourage the flow of water away from structures during construction and after placement of fill.

The subgrade should be prepared by excavating to the design grade and proof rolling with a heavy roller or other equipment to verify uniformity of the subgrade. Soft material should be excavated an additional 600 mm, minimum and the area backfilled with compacted pit run gravel. Pit run gravel should be compacted to a minimum 100% of its maximum dry density to minimize the potential for differential settlement.

6.2 Augered Cast-in-Place Concrete Piles

Augered cast-in-place concrete piles may be designed to develop their capacity on the basis of skin friction or end bearing, but not both. Piles developing their capacity on the basis of end bearing will undergo larger settlement than piles developing their capacity on the basis of skin friction.

The minimum length of pile is 6 m to ensure that a pile can resist any potential uplift forces associated with adfreeze forces or heave associated with an increase in water content.

Significant seepage and sloughing was not noted during drilling; however, some seepage can be expected from sand or sandy layers or fractures and fissures within the till, which may necessitate the use of temporary sleeving. Cobbles and boulders are common in glacial till, and should be expected in excavations. Cobbles and boulders that are smaller than the shaft diameter can be extracted on auger flights. Coring will be required to penetrate larger boulders.

For pile groups, a minimum centre to centre spacing of 2.5 times the pile diameter is recommended. A group efficiency of 1.0 is recommended for the determination of group capacity.

Ultimate skin friction values for concrete pile foundations are summarized in Table 6.2. The skin friction contribution of the upper 2 m of a pile constructed below finished grade and the capacity of any portion of the pile penetrating fill, should be ignored in the determination of pile capacity. Geotechnical resistance factors for analyses have been provided in Table 6.1.

Table 6.2 Summary of Ultimate Pile Design Parameters – Concrete Piles

Depth (m)		Soil	Ultimate Skin Friction	Ultimate End Bearing
From	To		(kPa)	(kPa)
0	2	Clay /Till	0	—
2	8	Clay / Till	68	1,050
8	12	Till	96	2,200

6.2.1 Belled Piles

Piles can also be under-reamed or belled, with capacity calculated on the basis of end bearing only. The bell must be constructed at least 2.5 bell diameters below grade to allow development of capacity. The ultimate end bearing value bells have been summarized in Table 6.2. Geotechnical resistance factors for analyses have been provided in Table 6.1.

Bells should be left open for a minimum amount of time prior to concreting to minimize the risk of heave of the bottom associated with groundwater levels being above the base of the bell and from unloading associated with excavation.

6.2.2 Uplift Capacity

Uplift capacity of an augered cast-in-place concrete pile can be calculated on the basis of the ultimate skin friction values provided in Table 6.2 and the geotechnical resistance factors in Table 6.1. The weight of the pile can be included in the calculation. The end bearing component should not be included in the calculation of uplift capacity. The uplift capacity of a pile group will be the lesser of the sum of the uplift resistance of the piles in the group or the sum of the resistance mobilized on the surface perimeter of the group using the ultimate skin friction values provided in Table 6.2 and the geotechnical resistance factors in Table 6.1, plus the effective weight of the soil and piles enclosed within this perimeter.

6.2.3 Negative Skin Friction

Piles installed through a significant thickness of relatively unconsolidated fill material should be designed to accommodate negative skin friction which can develop as the fill settles. Pile capacity should consider negative skin friction plus dead load. It should not consider live load. Values of negative skin friction will be the same as ultimate skin friction values provided in Table 6.2.

6.2.4 Confirmation of Design

Pile design parameters or pile capacity may be confirmed by dynamic or static pile load tests in accordance with the following ASTM standards:

- ASTM D1143 Standard Test Methods for Deep Foundations Under Static Axial Compressive Load.
- ASTM D4945 Standard Test Method for High-Strain Dynamic Testing of Piles.

As illustrated in Table 6.1, conducting a static load test will allow the use of a higher resistance factor to determine pile capacity. Pile load tests may be conducted on production or prototype piles. The design pile capacity may influence the selection of the type of pile tested. If the design capacity of the production pile is high, it may be more economical to test a lower capacity prototype pile to confirm design parameters since it is desired to fail the pile during testing to assess ultimate design parameters. Alternately, the pile can be tested to confirm design capacity.

6.2.5 Settlement

Settlement of augered cast-in-place concrete piles developing their capacity on the basis of skin friction is expected to be less than about 5 mm to 8 mm. Piles developing their capacity on the basis of end bearing will settle approximately 15 mm to 25 mm. The amount of settlement for an end bearing pile will depend upon the amount of loosened material left on the base of the pile after construction of the bell. Bells must be well cleaned to limit settlement to these values. A suggested definition of ‘clean’ for specifications is as follows:

- The average thickness of loose soil at the base of the bell should be no more than 15 mm and should cover less than 50% of the base of the bell, with no more than 40 mm of loose spoil material at any point on the bearing surface.
- No more than 50 mm of water should be allowed to accumulate at the base of the bell prior to concreting.

6.3 Driven Steel or Timber Piles

Driven steel or timber piles are a suitable option for this site. Driven steel or timber piles may be designed to develop their capacity on the basis of skin friction and end bearing as the end bearing resistance is fully mobilized by the driving process.

6.3.1 Design

Ultimate values for skin friction and end bearing for driven piles have been provided in Table 6.3. Geotechnical resistance factors are provided in Table 6.1.

For H-section piles, the box area calculated as the flange width times the depth may be used in the calculation of end bearing. It is assumed that a plug of soil will form between the flanges as the pile is driven. Closed end pipe piles can be utilized. For open end piles, the creation of a soil plug can be assumed, which allows the use of the plugged end area for calculation of end bearing.

Table 6.3 Summary of Ultimate Pile Design Parameters – Driven Piles

Depth (m)		Soil	Ultimate Skin Friction		Ultimate End Bearing
From	To		Steel (kPa)	Timber (kPa)	(kPa)
0	2	Clay /Till	0	0	—
2	8	Clay / Till	60	110	1,050
8	12	Till	96	110	2,200

* Minimum pile depth is 8 m.

As much as 75% to 85% of the pile capacity will be developed in skin friction. This proportion will vary with the length of the pile, since the skin friction component will increase with increased pile length, while the end bearing component remains relatively constant.

The skin friction contribution of the upper 2 m of pile below finished grade or of any fill material should be ignored in the determination of pile capacity. The minimum length of pile is 8.0 m.

As the pile is struck with a hammer with a quantity of kinetic energy, the pile will penetrate the soil a distance referred to as the set. The size of the set for a given quantity of energy is related to the soil resistance; the smaller the set, the greater the soil resistance. Thus, for a fixed energy in a hammer blow, a smaller set implies a greater pile capacity.

The length of a pile required to support the desired load can be estimated using skin friction and end bearing capacity for a specific pile type with a known cross section. A hammer and driving system must then be selected so that the pile can be driven to the design depth.

The tensile and compressive stresses in the pile during driving must then be analyzed to ensure that the pile is capable of being driven with the selected hammer to the required depth without failing the pile. If analyses indicate that compressive or tensile stresses are excessive, the hammer energy can be reduced.

Additionally, if the predicted blow count to achieve the desired penetration is excessive, defined to be more than about 400 blows/m, a more powerful hammer will be required.

If both the blow count and the compressive stresses are excessive, a pile with a larger section should be selected.

A refusal criteria can be selected to reduce the risk of damage to the pile during driving. However, unless confirmed by dynamic load testing, the pile must be driven to the design depth so that it will possess adequate capacity. ‘Refusal’ is dependent on the hammer energy. If the hammer selected does not possess adequate energy to drive the pile to the design depth, the pile may be observed to 'refuse'. This will not be an indication that the pile will possess adequate capacity. Practical refusal can be considered to be 400 blows/m, or 2.5 mm/blow.

In general, the hammer energy required to drive a pile will depend on the required pile capacity. As an approximate guide, the hammer-rated energy for driving steel H and pipe piles should be limited to a value of 6 x 10^6 J times the cross sectional area of the pile to reduce the risk of damage, or about 1.6 x 10^6 J times the pile head diameter (in metres) for timber.

Piles should be driven continuously once started, since setup, which is generally associated with dissipation of excess pore water pressures with time, may unnecessarily increase the driving effort. In some circumstances, it may be impossible to remobilize a pile that has been left for a few hours. The ratio of the mobilized skin friction while driving to the long term mobilized skin friction of 0.5 will be appropriate for clay soil. No reduction factor is required when considering the end bearing component.

For pile groups, a minimum centre to centre spacing of 2.5 times the pile diameter is recommended. A group efficiency of 1.0 is recommended for the determination of group capacity.

Uplift capacity of a driven steel or timber pile can be calculated on the basis of the ultimate skin friction values provided in Table 6.3 and the geotechnical resistance factors in Table 6.1. The weight of the pile can be included in the calculation. The end bearing component should not be included in the calculation of uplift capacity. The uplift capacity of a pile group will be the lesser of the sum of the uplift resistance of the piles in the group or the sum of the resistance mobilized on the surface perimeter of the group using the ultimate skin friction values provided in Table 6.3 and the geotechnical resistance factors in Table 6.1, plus the effective weight of the soil and piles enclosed within this perimeter.

6.3.2 Settlement

Settlement of a driven steel or timber pile is expected to be less than 5 mm to 10 mm.

6.3.3 Quality Assurance Testing

Load testing of piles is the most positive method to determine load carrying capacity and can form a fundamental part of the pile design process. Load testing can be conducted as part of the design process, during construction as proof tests, or as part of the quality assurance program for construction of the foundation.

The results of a pile load test can be used to evaluate the ultimate load carrying capacity of a pile and its load-settlement behaviour. It also provides a means to verify design assumptions.

Pile load tests may be conducted on production or prototype piles. The design pile capacity may influence the selection of the type of pile tested. If the design capacity of the production pile is high, it may be more economical to test a lower capacity prototype pile to confirm design parameters since it is desired to fail the pile during testing to assess ultimate design parameters.

If dynamic load testing is incorporated into the quality assurance testing program, the design pile capacity can be confirmed for the design pile length or depth of installation. Piles that cannot be driven to the design depth considering a practical refusal criteria of about 2.5 mm/blow should not be accepted unless it can be demonstrated by analysis that the pile has adequate capacity considering the hammer energy and pile set.

The number of pile load tests conducted will depend on the number of piles being driven and the variability of results obtained during construction. Additional pile load tests are suggested if test results indicate variable driving conditions due to the equipment used or subsurface conditions. Piles should be restruck to assess set. In general, the acceptance criterion can be an average capacity of no less than 85% of the estimated ultimate capacity for the tested piles.

6.4 Helical Piles

Helical piles may be a suitable foundation for this Site. The lateral load carrying capacity of the helical pile should be assessed by a structural consultant.

6.4.1 Design

The basis for design of helical piles considers a component of end bearing and skin friction for the shaft of the pile for a single helix pile or skin friction for a cylinder of soil between helixes for a multiple helix pile. The bottom of the pile should be installed below the depth of freezing, and preferably at least 6 m below ground surface to resist potential frost heave. The minimum depth of penetration is reduced when compared to a driven pile due to the presence of the helix that will provide resistance in uplift. The skin friction component of capacity is typically ignored for a pile shaft diameter less than 100 mm.

The installation torque will be dependent on soil conditions, the ultimate capacity of the pile and its shaft diameter. The installation torque can be estimated using the equation:

$$T = Q_{all} / K_T$$

where: Q_{all} = allowable capacity of the pile in N or lb,
T = torque in N-m or ft-lb, and
K_T = empirical factor.

Values for K_T will likely range from 3/ft to 20/ft for capacity in lb and torque in ft-lb, or 10/m to 33/m for T in N-m. For shafts of about 90 mm diameter, the value for K_T will be about 7/ft (23/m), with K_T decreasing to about 3/ft (10/m) for shaft diameters approaching 200 mm.

For a single helix pile installed into clay or till, the ultimate pile capacity, Q_{ult} (kN) can be estimated using the following equation:

$$Q_{ult} = q_{end,ult}\ A + \pi\ d\ q_{skin,\ ult}\ (H - 2)$$

where: $q_{end,ult}$ = ultimate bearing capacity as shown in Table 6.3,
$q_{skin,\ ult}$ = ultimate skin friction for steel as shown in Table 6.3,
A = area of helix (m^2),
d = shaft diameter (m), and
H = depth from ground surface to helix (m).

6.4.2 Settlement

Settlement of a properly designed and installed helical pile is expected to be less than 25 mm.

6.5 Lateral Loads on Piles

The lateral load carrying capacity and deflection of a pile subjected to a lateral load is dependent on the stiffness of the pile and soil strength. The stiffness of a pile can be calculated using well defined properties of steel or concrete, or both; however, the response of soil under loading is subject to some variability. The best method to evaluate the performance of a pile subjected to a lateral load is a well-designed and executed lateral load test based on ASTM D3966-07, Standard Test Methods for Deep Foundations under Lateral Loads.

The performance of laterally loaded augered cast-in-place concrete piles may be analyzed using the software application LPILE. This program computes deflection, shear, bending moment and soil response with respect to depth in a nonlinear soil. Soil behaviour is modelled with *p-y* curves that are generated by the software following published recommendations for various types of soils. These relationships consider the relationship between undrained shear strength and soil modulus, as well as strain at 50% of the maximum stress.

The lateral load carrying capacity of pile groups will depend on the pile spacing and orientation of the piles. In general, pile spacing should be at least 2.5 pile diameters. For preliminary analyses, an efficiency of 0.70 may be used for a group loaded parallel to the piles. This can be increased to 0.90 for loading perpendicular to the piles. Specific configurations with specific lateral, vertical or moments can be analyzed using software applications such as GROUP. Analyses for this site can assume soil properties shown in Table 6.4. Analyses can be undertaken once the magnitude of applied pile loads has been assessed.

Table 6.4 Soil Properties for Analyses of Laterally Loaded Piles Using LPile

Soil Strata	Depth (m)	Undrained Shear Strength (kPa)	Effective Unit Weight (kN/m^3)
Clay (BH102)	0 to 2	150	15.7
Clay (BH102)	2 to 7.6	150	5.7
Till (BH101, BH103)	0 to 2	150	16.1
Till (BH101, BH103)	2 to 7.6	150	6.1
Till	7.6 to 12.2	250	9.0

Piles can be analyzed for a fixed or free pile head condition. The fixed condition maintains a zero slope at the top of the pile and the pile head is allowed to translate. A free pile head condition allows rotation at the head of the pile.

The performance of the pile subjected to a lateral load will be most sensitive to the properties of soil near the top of the pile. There will be a significant difference between deflection associated with a fixed head and a free head condition, with a reduction in pile head deflection for an increased pile diameter. Pile head deflection and moment is not expected to vary significantly for a reasonable range of applied vertical load.

Pile behaviour may be approximated using analysis that incorporates a coefficient of horizontal subgrade reaction, k_s. The coefficient of horizontal subgrade reaction is a rough approximation at best and includes a high degree of uncertainty due to the influence of stress level, pile geometry and empirical nature of expressions used to derive these values. Values for k_s are summarized in Table 6.5.

Table 6.5 - Coefficient of Horizontal Subgrade Reaction, k_3

Soil	Depth (m)	k_3 (kN/m^3)
Clay/Till	0 to 8	10,000/B
Till	7.6 to 12.2	15,000/B

B – Pile Diameter (m)

7.0 General Foundation Recommendations

7.1 Shallow Spread Footing

The following recommendations are made for a shallow spread footing:

- Significant heave may occur if foundations are founded on high plasticity clay.
- Footings constructed above a depth of about 2.5 m may be prone to frost heave. Insulation may be incorporated into footing design to minimize frost related issues. In general, the bearing surface for a footing should be prepared by removing any pockets of soft soil or soft fill to a uniform bearing surface. The surface must be maintained in an undisturbed state. The excavated surface can be protected with a mudslab placed within 24 hours of completion of excavation.
- Seepage should be expected in footing excavations, especially below a depth of about 2.2 m. Sloughing may be encountered in sand or sandy layers within the till. The requirement for pumping of water from excavations should be expected.
- Overexcavated areas may be filled with a lean concrete mix or with a well graded pit run gravel that conforms to the Recommended Specifications for Granular Materials appended to this report. The material must be compacted to 100% of its maximum dry density determined in accordance with ASTM D698-00a, Standard Test Methods for Laboratory Compaction Characteristics of Soil Using Standard Effort [12,400 ft lbf/ft^3 (600 kN m/m^3)].
- The foundation must be adequately reinforced to distribute the applied loads and also have sufficient stiffness to distribute local overstresses.
- The minimum footing width is 450 mm.
- A shallow spread footing constructed as specified above may be designed on the basis of an ultimate bearing capacity of 500 kPa, using the geotechnical resistance factors shown in Table 6.1. For serviceability criteria, bearing capacity can be selected on the basis of settlement or differential settlement as described in previous sections.

7.2 Augered Cast-in-Place Concrete Piles

Augered cast-in-place piles developing their load carrying capacity on the basis of skin friction can be considered for structures at this site. If additional capacity is required, piles may be designed on the basis of end bearing. Our specific design criteria and recommendations for augered cast-in-place piles are as follows:

- Foundation loads may be supported on piles designed as straight shafts, developing load carrying capacity on the basis of skin friction only. Values for ultimate skin friction value are summarized in Table 6.2. The geotechnical resistance factors have been summarized in Table 6.1. The skin friction contribution of the upper 2 m (minimum) below finished grade and any fill material should be ignored in the calculation of pile capacity.
- On the basis of observations during the field investigation, some seepage may be encountered from sand or sandy lenses, or fractures and fissures in the till below 2.2 m. Temporary sleeving may be required for piling. Cobbles and occasional boulders can be expected in the till strata. Concrete should be placed in dry shafts within 2 hrs of excavation to minimize softening of the clay or clay till, which can reduce pile capacity and squeezing of soil, which can result in necking. Should seepage or sloughing occur, the hole should be pumped dry before concreting.

- It is suggested that the aspect ratio of a pile, defined as the ratio between length and diameter, should not exceed 30. This should ensure that good contact is maintained between the concrete and soil and that no voids are created.
- The use of water to facilitate excavation of piles should be avoided, since this will result in softening of the soil in contact with the concrete, reducing pile capacity. Inspection during construction is recommended to ensure compliance with specifications.
- Pile shafts must be adequately reinforced to withstand the imposed stresses. Pile reinforcement should extend at least 5 m below finished grade and not less than two thirds the pile length.
- If additional pile capacity is required, the piles may be belled out or expanded at the base. The bell must be constructed at least 2.5 bell diameters below grade to allow development of capacity. Belled piles constructed on very stiff to hard till will provide high capacity.
- The ultimate end bearing values for a well-constructed, machine cleaned bell have been provided in Table 6.2. Geotechnical resistance factors for analyses have been provided in Table 6.1.
- The base of the bell should be a minimum 500 mm below the top of the soil stratum upon which is designed to bear. This value and geotechnical resistance factors used for design may be revised on the basis of a pile load test.
- Inspection by qualified geotechnical personnel is necessary to ensure that end bearing piles constructed in this manner will be capable of developing these capacities. Skin friction along the shafts of belled piles may not be used in the design of end bearing piles.
- The average thickness of loose soil at the base of the bell should be no more than 15 mm and should cover less than 50% of the base of the bell, with no more than 40 mm of loose spoil material at any point on the bearing surface.
- No more than 50 mm of water should be allowed to accumulate at the base of the bell prior to concreting.
- The undrained shear strength of soil at the base of the bell may be verified using a pocket penetrometer, laboratory vane shear apparatus or other suitable tools. Soil at the base of the pile can be sampled using a thin walled tube sampler to ensure that no ‘build up’ is present.
- If bells are to be inspected by downhole personnel, piles should have a minimum shaft diameter of 700 mm to allow installation of a sleeve for safety in accordance with Occupational Health and Safety guidelines. Such piles should be inspected by a qualified Geotechnical Engineer or personnel under the supervision of a qualified Geotechnical Engineer to confirm the soil strength parameters and approve the piling construction methods. Inspection will be limited to that area within the shaft so that personnel remain within the sleeved area.
- Bells should be constructed with a sideslope not less than 45° and preferably 60° from the horizontal. The base of the bells should be excavated vertically a minimum of 200 mm to allow adequate load transfer to the soil. The base of the bell must be excavated into undisturbed foundation soils of adequate capacity as described in previous sections to carry the design loads. The area around the shaft extension should be carefully cleaned and the edges of the shaft extension bevelled to reduce the risk of stress concentrations within the finished bell.

7.3 Driven Steel or Timber Piles

Driven piles consisting of steel or timber may be considered to support the proposed structure. For preliminary purposes, our specific design recommendations for a driven pile foundation system are as follows:

- If practical, we recommend that test piles be driven and tested prior to the ordering of construction piles. The test should at least consist of driving the piles, allowing it to sit overnight, and then redriving it the following day. Piles should also be restruck at least two weeks after installation. The pile should not have any further penetration. Group action must be considered when evaluating the results of the test.
- The capacity of the piles may be estimated on the basis of skin friction and end bearing.
- For preliminary design, ultimate values for skin friction and end bearing are summarized in Table 6.3. Geotechnical resistance factors have been provided in Table 6.1. The skin friction contribution of the upper 2 m of pile below finished grade and any fill material should be ignored in the determination of pile capacity.
- Required pile lengths may vary greatly, particularly in pile groups; therefore, the need for qualified inspection, testing of piles and suitable specifications is paramount. The minimum recommended pile spacing for pile groups is 2.5 times the nominal pile width.
- Piles should be driven continuously, once started, to ensure that setup associated with dissipation of excess pore water pressure does not unnecessarily increase the driving effort.
- As an approximate guide, the hammer-rated energy for driving steel H and pipe piles should be limited to a value of 6×10^6 J times the cross sectional area of the pile for steel, or 1.6×10^6 J times the cross sectional area of the pile for timber. Practical refusal can be considered to be 400 blows/m, or 2.5 mm/blow.

8.0 Floor Considerations

8.1 Grade Supported Floors

Floors constructed on high plasticity clay will be subject to significant vertical movement associated with changes in soil moisture, which will result in cracking and some loss of utility. If this level of performance is not acceptable, mitigative measure should be explored. Alternately, the floor can be structurally supported. Our recommendations for a grade supported floor slab are as follows:

- The subgrade soil below the proposed floor slab should be excavated to undisturbed soil. Construction on fill material of unknown quality and composition can result in uneven settlement or heave. All topsoil must be removed from the site during subgrade preparation for the grade supported floor slab. Care must be exercised to remove all loose soil and debris. Soft, wet areas, which do not have sufficient trafficability for construction purposes, may be further excavated and replaced with a pit run sand or gravel which complies with the attached specifications.
- The excavated subgrade should be uniformly compacted to 98% of its maximum dry density determined in accordance with ASTM D698-00a, Standard Test Methods for Laboratory Compaction Characteristics of Soil Using Standard Effort [12,400 ft-lbf/ft^3 (600 kN-m/m^3)]. The water content of the subgrade should be adjusted to optimum water content ±2%.

- Place a crushed base course which complies with the specifications given in the Recommended Specifications for Granular Materials appended to this report for Type 32 or 33 base course.
- Compact the base course to a minimum average 98% of its maximum dry density for four (4) consecutive tests, with no single test less than 96%, as determined in accordance with ASTM D698-00a, Standard Test Methods for Laboratory Compaction Characteristics of Soil Using Standard Effort [12,400 ft-lbf/ft^3 (600 kN-m/m^3)]. Water may be used as an aid to compaction and vibratory compaction equipment is recommended.
- A layer of polyethylene sheeting 150 μm (minimum) thick should be placed between the granular base and the concrete slab to deter the migration of moisture through the floor and loss of moisture from freshly placed concrete.
- The floor must be structurally isolated from other building elements, service lines and appurtenant structures to prevent stresses caused by floor movement from being transmitted to these elements.
- Positive site drainage around the building and control of roof drainage away from the building reduces the risk of volume change in grade supported floors.

8.2 Structurally Supported Floor

The following procedures are recommended for the construction of a structurally supported floor:

- The subgrade should be positively graded to a sump to remove water, which may inadvertently pond beneath the floor.
- Migration of moisture from the soil should be prevented by installing 150 m (minimum) thick polyethylene vapour barrier covered with 50 mm of sand.
- Floors designed as a structurally supported system with a crawl space between the floor and the subgrade should have some provision to ventilate the crawl space, particularly during the summer months.
- As an alternative to a crawl space, the floor may be cast upon waxed cardboard carton ‘void form’ that is designed to degrade following the placement of the concrete. The cardboard cartons must have a strength sufficient to support the fresh concrete until it has sufficient strength to be self-supporting. Great care is required during construction of such floor systems to ensure that the collapse of the cartons does not take place, resulting in a grade supported slab. Careful inspection of these floors during construction is required to ensure that the void does not collapse during the placement of the floor. Further, care must be taken during selection of ‘void form’ used. Materials which depend upon biologic degradation should be avoided.

9.0 Excavation Considerations

The stability of cut slopes and the stability of any adjacent structure must be considered for any excavations on the Site. The anticipated sideslopes for the excavation will depend on the soil texture, water content, and length of time that the excavation is left open.

Seepage should be expected in excavations below a depth of 2.2 m. Some sloughing may occur from sand or sandy lenses within the till. The potential requirement for pumping of water from excavations should be considered.

Excavations should be performed in compliance with provincial safety regulations. Soil in this area will be a type 3 soil as defined by Occupational Health and Safety regulations. In general, excavations should be no steeper than about 1 horizontal to 1 vertical (1:1). Although excavations through these materials may stand in the short term at steeper angles, over steepened slopes will slough and collapse if they are left open for long periods of time or if water is allowed to infiltrate. Excavation conditions must be carefully monitored, and slopes flattened during construction if conditions warrant. Failure may be sudden and may endanger personnel and equipment working in the vicinity.

Sideslopes may have to be adjusted in the field as excavation progresses, depending upon conditions encountered. Seepage could contribute to erosion of a slope, and slopes should be monitored and cut back as required during construction. Continuous inspection is recommended since slope failure could be sudden.

All loose material on the sides of the excavation should be trimmed. The excavation should be left open for the minimum amount of time required for construction. Some loss of strength in the soil can be expected with the passing of time, resulting in sloughing and local slope failures.

As described in Occupational Health and Safety Regulations, a competent worker should be stationed on the surface to alert any worker in the excavation about the development of any potentially unsafe conditions. Machinery and heavy equipment should not be allowed closer to the excavation than one half of the depth of the excavation, unless precautions are implemented to ensure that workers in the excavation are safe. Spoil material should not be piled closer than 3 m from the edge of the excavation and with sideslopes no steeper than 1:1.

Infiltration of water into the soil around the excavation can result in loss of strength and collapse of the excavation walls. It is recommended that workers not be in the excavation during rainfall and that excavation walls be carefully inspected for cracking, sloughing, and potential failures after rainfall before work continues in the excavation.

10.0 Underground Walls

It is recommended that the underground walls should be designed to withstand the lateral earth pressure (p) at any depth (H) as estimated by the following expression:

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

where: γ = unit weight of the wall backfill, provided in Table 5.2
q = the vertical pressure of any surcharge acting at ground surface near the wall
K = the active earth pressure coefficient provided in Table 5.2 for a wall that is allowed to rotate, and the at rest earth pressure coefficient for a rigid wall

This expression assumes that the wall will be backfilled with a free draining granular backfill and will not be subject to build up of water pressure behind the wall. If effective wall drainage cannot be guaranteed, full hydrostatic pressure, which may act on the wall, must be considered in the design.

Free draining backfill materials should be placed adjacent to the exterior underground walls. Free draining means that the granular material should be well graded and have less than 3 percent passing the 75 μm sieve. The upper 0.6 m of backfill should consist of local compacted soil or the surface must be covered with some other suitable impermeable material. The ground surface should be contoured away from the building to further discourage the entry of surface runoff into the backfill. Regardless of the type of backfill used behind the wall, it is recommended that the wall be effectively damp-proofed to prevent migration of moisture through the concrete. Damp-proofing also aids in reducing the rate of deterioration of the concrete due to chemical attack and weathering.

11.0 Closure

This report was prepared by Clifton Associates Ltd. for the use of the RCMP and their agents for specific application to the proposed RCMP New Detachment building in Onion Lake, Saskatchewan. The material in it reflects Clifton Associates Ltd. best judgment available to it at the time of preparation. Any use which a third party makes of this report, or any reliance on or decisions to be made based on it, are the responsibility of such third parties. Clifton Associates Ltd. accepts no responsibility for damages, if any, suffered by any third party as a result of decisions made or actions based on this report.

This report has been prepared with generally accepted engineering practices common to the local area. No other warranty, expressed or implied, is made.

Our conclusions and recommendations are preliminary and based upon the information obtained from the referenced subsurface exploration. The borings and associated laboratory testing indicate subsurface and groundwater conditions only at the specific locations and times investigated, only to the depth penetrated and only for the soil properties tested. The subsurface and groundwater conditions may vary between the boreholes and with time. The subsurface interpretation provided is a professional opinion of conditions and not a certification of the site conditions. The nature and extent of subsurface variation may not become evident until construction or further investigation. If variations or other latent conditions do become evident, Clifton Associates Ltd. should be notified immediately so that we may re-evaluate our conclusions and recommendations. Although subsurface conditions have been explored, we have not conducted analytical laboratory testing on samples obtained nor evaluated the site with respect to the potential presence of contaminated soil or groundwater. The enclosed report contains the results of our investigation as well as certain recommendations arising out of such investigations. Our recommendations do not constitute a design, in whole or in part, of any elements of the proposed work. Incorporation of any or all of our recommendations into the design of any such element does not constitute us as designers or co-designers of such elements, nor does it mean that the design is appropriate in geotechnical terms. The designers of such elements must consider the appropriateness of our recommendations in light of all design criteria known to them, many of which may not be known to us. Our mandate has been to investigate and recommend which we have completed by means of this report. We have had no mandate to design, or review the design, of any elements of the proposed work and accept no responsibility for such design or design review.

Clifton Associates Ltd.

Richard T Yoshida PEng

Association of Professional
Engineers and Geoscientists of Saskatchewan
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Appendix A

Clifton Associates

Drawings

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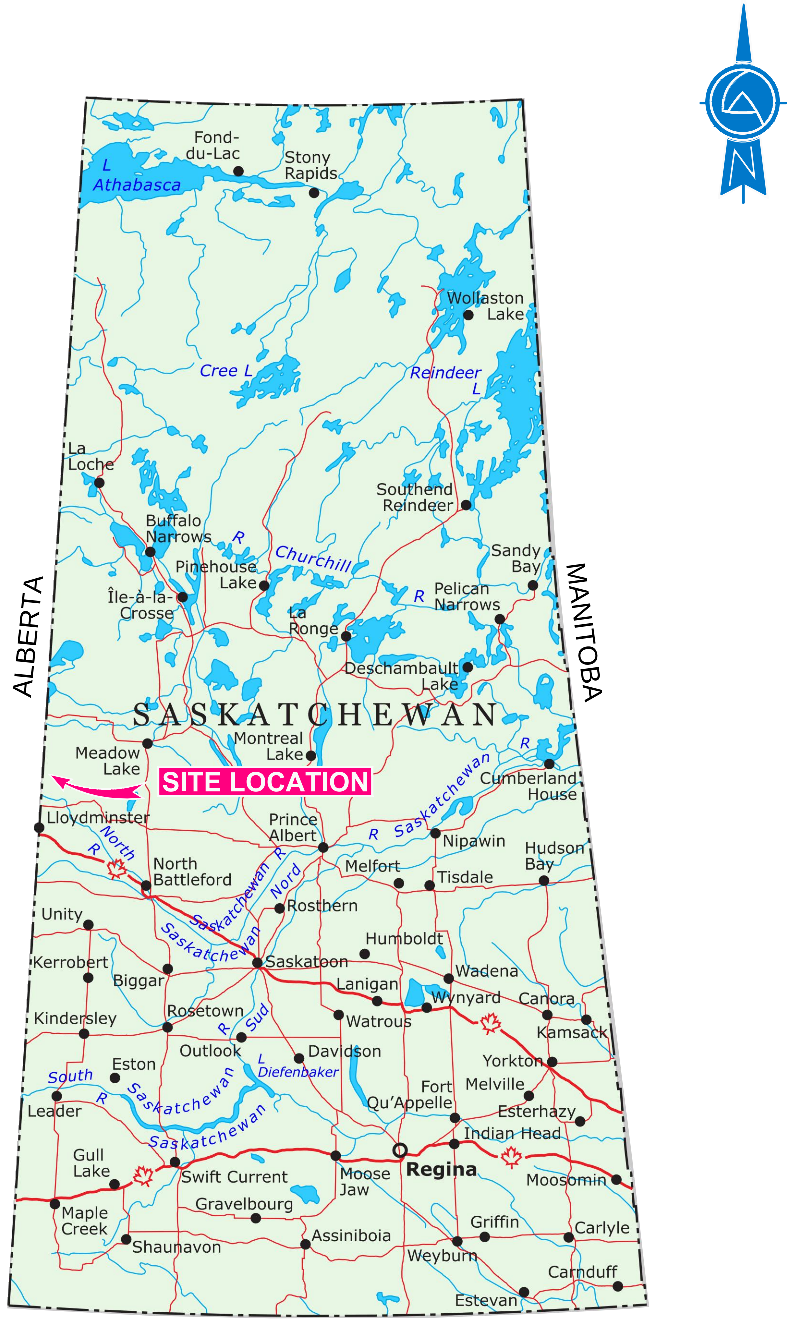
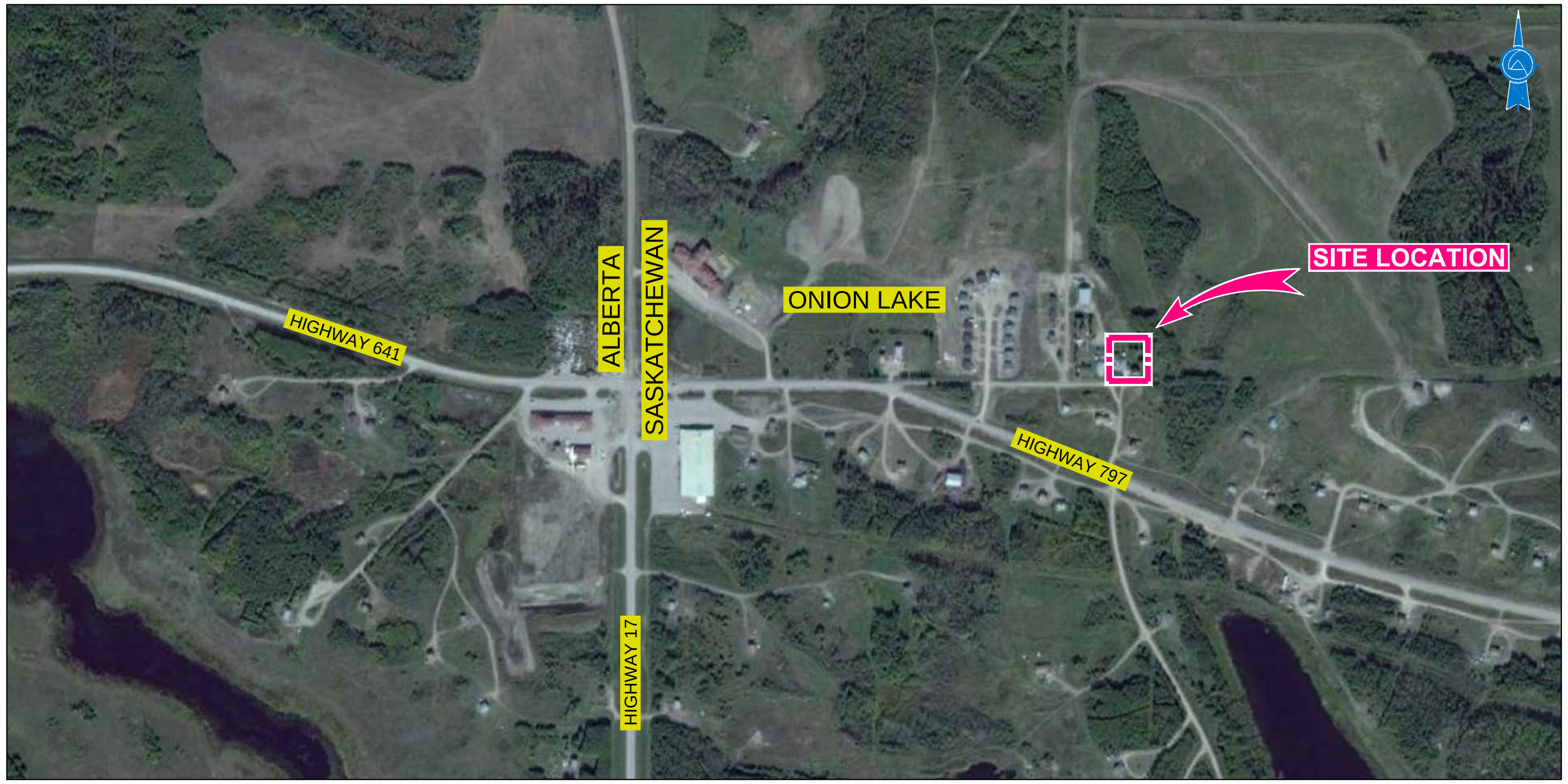


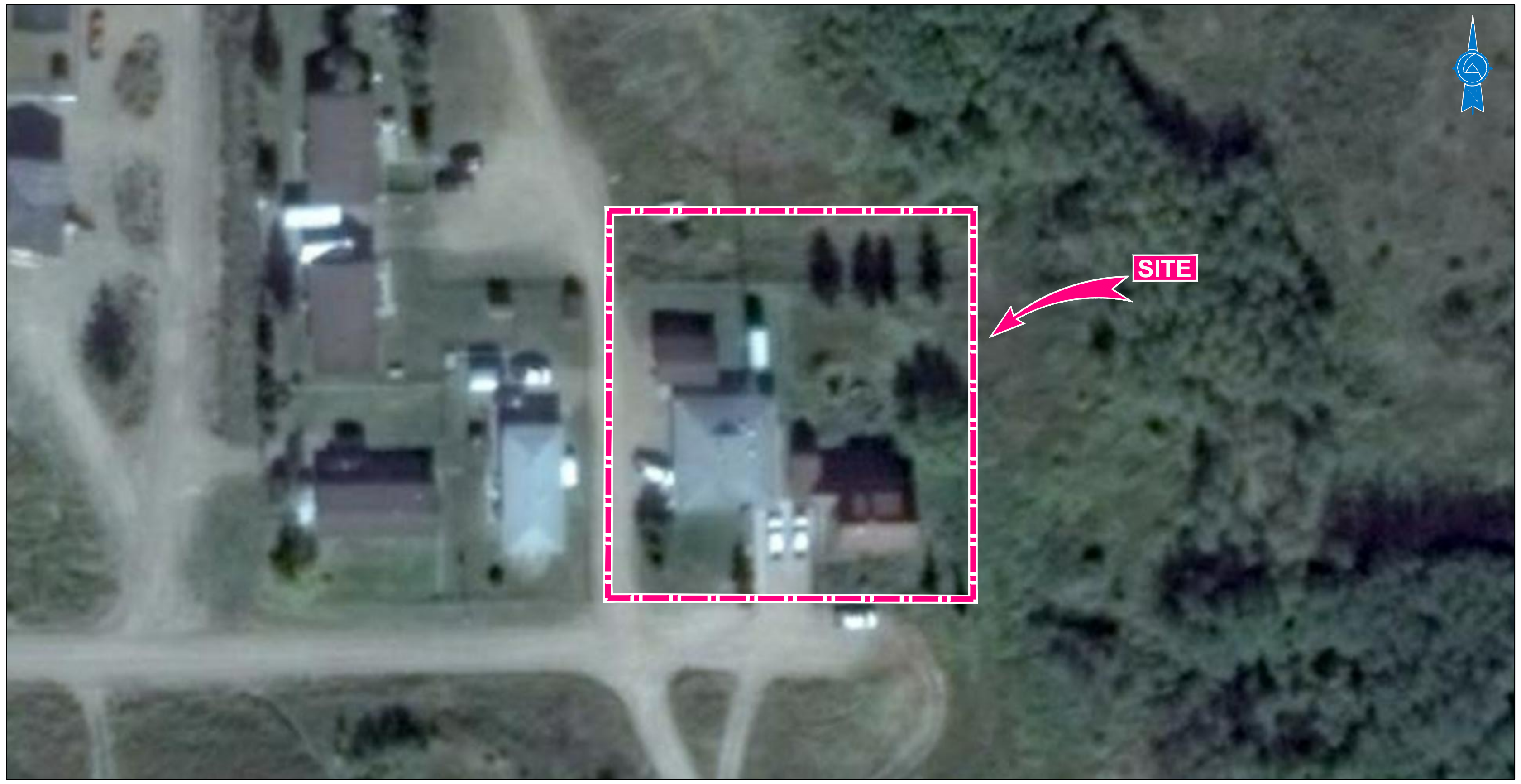
KEY PLAN

SCALE 1:7,500,000



LOCATION PLAN

SCALE: NTS



SITE PLAN

SCALE: 1:1000

LEGEND

SITE LOCATION —·—·—

NOTES:

1. LOCATION AND SITE PLAN IMAGES PROVIDED BY GOOGLE EARTH PRO, 2011.
2. KEY PLAN IMAGE PROVIDED BY GOVERNMENT OF CANADA; NATURAL RESOURCES CANADA; EARTH SCIENCES SECTOR; THE ATLAS OF CANADA, 2001.
3. LEGAL DESCRIPTION: SW 05-55-27 W3M, ONION LAKE, SASKATCHEWAN.

DRAWING REVISIONS

REV	DESCRIPTION	BY	DATE

ENGINEER

Clifton Associates

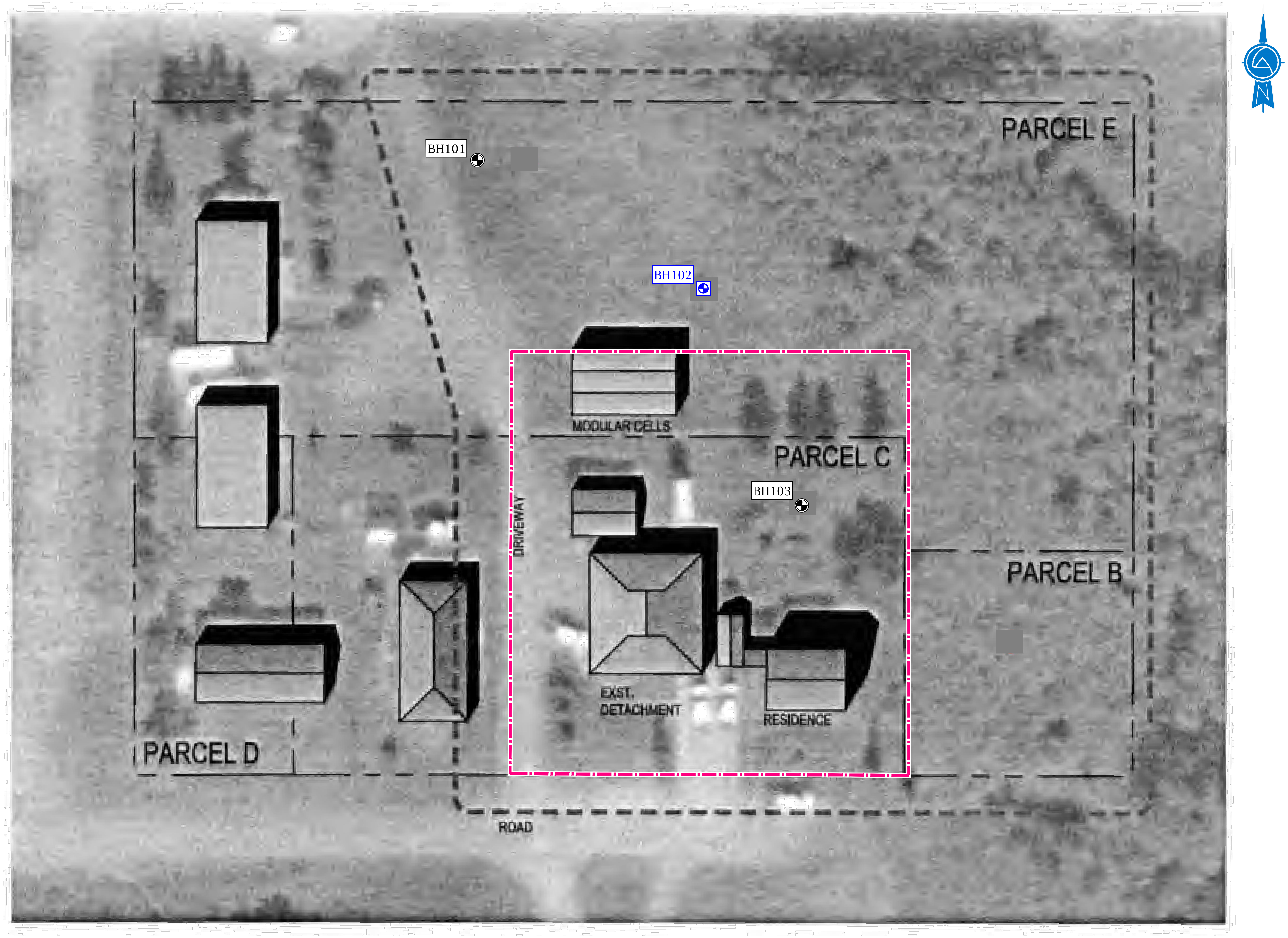
CLIENT
RCMP

PROJECT
GEOTECHNICAL INVESTIGATION,
NEW DETACHMENT
ONION LAKE, SK

TITLE
SITE LOCATION PLAN

DESIGNED JW	SCALE AS SHOWN	DATE 2015-01-20
DRAWN SP	PROJECT NO. S2091	DWG NO. S2091-01
CHECKED JLO	FILE NO. S2091-01	SHEET NO.

G:\Stoon_Proj\S2000's\S2090 – S2099\S2091 – RCMP – Onion Lake – Geotech\500_Data\S2091\!CURRENT DRAWINGS\S2091-01.dwg, 01/20/2015 8:51:46 AM



LEGEND

BOREHOLE	⊕
MONITORING WELL	⊕
SITE LOCATION	–·–·–

NOTES:
1. IMAGE PROVIDED BY CLIENT.
2. LEGAL DESCRIPTION:
SW 05–55–27 W3M,
ONION LAKE, SASKATCHEWAN.

DRAWING REVISIONS

REV	DESCRIPTION	BY	DATE

ENGINEER
Clifton Associates

CLIENT
RCMP

PROJECT
GEOTECHNICAL INVESTIGATION,
NEW DETACHMENT
ONION LAKE, SK

TITLE
BOREHOLE LOCATION PLAN

DESIGNED JW	SCALE 1:500	DATE 2015-01-20
DRAWN SP	PROJECT NO. S2091	DWG NO. S2091-02
CHECKED JLO	FILE NO. S2091-02	SHEET NO.

G:\Stoon_Proj\S2000's\S2090 – S2099\S2091 – RCMP – Onion Lake – Geotech\500_Data\S2091\!CURRENT DRAWINGS\S2091–02.dwg, 01/21/2015 11:32:20 AM

Appendix B

Clifton Associates
Borehole Logs and Laboratory Test Data

Clifton Associates

Regina Office
340 Maxwell Crescent
Regina, SK S4N 5Y5

T (306) 721-7611
F (306) 721-8128

regina@clifton.ca
www.clifton.ca

Soil Descriptive Terms

A soil description for geotechnical applications includes a description of the following properties:

- texture
- color, oxidation
- consistency and condition
- primary and secondary structure

Texture

The soil texture refers to the size, size distribution and shape of the individual soil particles which comprise the soil. The Unified Soil Classification System (ASTM D2487-00) is a quantitative method of describing the soil texture. The basis of this system is presented on the following page. The following terms are commonly used to describe the soil texture.

Particle Size (ASTM D2487-00)

Boulder	300 mm plus
Cobble	75 – 300 mm
Gravel	4.75 – 75 mm
Coarse	19 – 75 mm
Fine	4.75 – 19 mm
Sand	0.075 – 4.75 mm
Coarse	2 – 4.75 mm
Medium	0.425 – 2 mm
Fine	0.075 – 0.425 mm
Silt and Clay	Smaller than 0.075 mm

Relative Proportions (CFEM, 4th Ed., 2006)

Trace	1 – 10 %
Some	10 - 20 %
Gravelly, sandy, silty, clayey, etc.	20 – 35 %
And	>35 %
Gravel, Sand, Silt, Clay, etc.	35% and main fraction

Gradation

Well Graded	Having a wide range of grain sizes and substantial amount of all intermediate sizes.
Uniform or Poorly Graded	Possessing particles of predominatly one size.
Gap Graded	Possessing particles of two distinct sizes.

Particle Shape

Angular	Sharp edges and relatively plane sides with unpolished face.
Subangular	Similar to 'angular' but have rounded edges.
Subrounded	Well-rounded corners and edges, nearly plane sides.
Rounded	No edges, has smoothly curved sides.
Also may be flat, elongated, or both.	

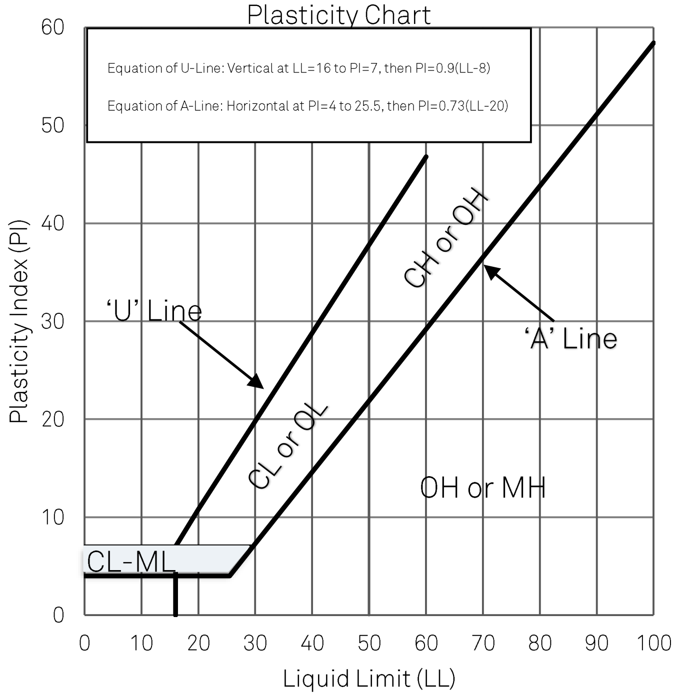
The term "TILL" may be used as a textural term to describe a soil which has been deposited by glaciers and contains an unsorted, wide range of particle sizes.

Colour and Oxidation

The soil color at its natural moisture content is described by common colors and, quantitatively, in terms of the Munsell color notation; (eg. 5Y 3/1). The notation combines three variables, hue, value and chroma to describe the soil color. The hue indicates its relation to red, yellow, green, blue and purple. The value indicates its lightness. The chroma indicates its strength of departure from a neutral of the same lightness. Departure of the soil color from a neutral color indicates the soil has been oxidized. Oxidation of a soil occurs in a oxygen rich environment where most commonly metallic iron, oxidizes and turns a neutral colored soil 'rusty' or reddish brown. Oxidized manganese gives a purplish tinge to the soil. Oxidation may occur throughout the entire soil mass or on fracture/joint/fissure surfaces.

Classification of Soils for Engineering Purposes

ASTM Designation D 2487-00 (Unified Soil Classification System)

Major divisions			Group Symbol	Typical Names	Classification Criteria			
Coarse-grained soils More than 50% retained on No. 200 sieve* (>0.075 mm)	Gravels More than 50% of coarse fraction retained on No. 4 sieve(4.75 mm)	Clean gravels <5% fines	GW	Well-graded gravel	If 15% sand add "with sand" to group name.	Classification on basis of percentage of fines: Less than 5% pass No. 200 sieve - GW, GP, SW, SP More than 12% pass No. 200 sieve - GM, GC, SM, SC 5 to 12% pass No. 200 sieve - Borderline classifications, use of dual symbols	$C_u = \frac{D_{60}}{D_{10}} \geq 4$; $C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ between 1 and 3	
			GP	Poorly graded gravel			Not meeting either Cu or Cc criteria for GW	
		Gravels with >12% fines	GM	Silty gravel			Atterberg limits below "A" line or PI less than 4	Atterberg limits plotting in hatched area are borderline classifications requiring use of dual symbols If fines are organic add "with orgnic fines" to group name
			GC	Clayey gravel			Atterberg limits on or above "A" line and PI > 7	
	Sands 50% or more of coarse fraction passes No. 4 sieve(<4.75 mm)	Clean sands <5% fines	SW	Well-graded sand	If 15% gravel add "with gravel to group name		$C_u = \frac{D_{60}}{D_{10}} \geq 6$; $C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ between 1 and 3	
			SP	Poorly graded sand			Not meeting either Cu or C ccriteria for SW	
		Sands with >12% fines	SM	Silty sand			Atterberg limits below "A" line or PI less than 4	Atterberg limits plotting in hatched area are borderline classifications requiring use of dual symbols If fines are organic add "with orgnic fines" to group name
			SC	Clayey sand			Atterberg limits on or above "A" line and PI > 7	
Fine-grained soils50% or more passes No. 200 sieve* (<0.075 mm)	Silts and Clays Liquid Limit <50%	Inorganic	ML	Silt	If 15 to 29% coarse-grained, add "with sand" or "with gravel" as appropriate. If > 30% coarse-grained, add "sandy" or "gravelly" as appropriate. Class as organic when oven dried liquid limit is < 75% of undried liquid limit.			
			CL	Lean Clay -low plasticity				
		Organic	OL	Organic clay or silt (Clay plots above 'A' Line)				
	Silts and Clays Liquid Limit >50%	Inorganic	MH	Elastic silt				
			CH	Fat Clay -high plasticity				
		Organic	OH	Organic clay or silt (Clay plots above 'A' Line)				
	Highly Organic Soils		PT	Peat, muck and other highly organic soils				

*Based on the material passing the 3 in.(75 mm) sieve, if field samples contain cobbles or boulders, add "with cobbles or boulder`s" to group name

Consistency and Condition

The consistency of a cohesive soil is a qualitative description of its resistance to deformation and can be correlated with the undrained shear strength of the soil. The condition of a coarse grained soil qualitatively describes the soil compactness and can be correlated with the standard penetration resistance (ASTM D1586-99).

Consistency of Ceohesive Soil (CFEM, 4th Edit., 2006)

Consistency	Undrained Shear Strength (kPa) (CFEM, 4th Edit., 2006)	Field Identification (ASTM D2488-00)
Very Soft	<12	Thumb will penetrate soil more than 25 mm.
Soft	12 – 25	Thumb will penetrate soil about 25 mm.
Firm	25 – 50	Thumb will indent soil about 6 mm.
Stiff	50 – 100	Thumb will indent, but penetrate only with great effort (CFEM).
Very stiff	100 – 200	Readily indented by thumbnail (CFEM).
Hard	>200	Thumb will not indent soil but readily indented with thumbnail.
Very Hard	N/A	Thumbnail will not indent soil.

Consistency of Coarse Grained Soil (CFEM, 4th Edit., 2006)

Compactness Condition	SPT N – Index (Blows/300mm)
Very Loose	0 – 4
Loose	4 – 10
Compact	10 – 30
Dense	30 – 50
Very Dense	Over 50

Moisture Conditions (ASTM D2488-00)

Description	Criteria
Dry	Absence of moisture, dusty, dry to touch.
Moist	Damp but no visible water.
Wet	Visible, free water, usually soil is below water table.

Structure

The soil structure is the manner in which the individual soil particles are assembled to form the soil mass. The primary soil structure is the arrangement of soil particles as originally deposited. The secondary soil structure refers to any rearrangement of the soil such as deformation and cracking which has taken place since deposition.

Primary Soil Structure (Depositional)

Geometry

Stratum	- A single sedimentary ‘layer’, greater than 10 mm in thickness, visibly separable from other strat by a discrete change in lithology and/or sharp physical break.
Homogeneous	- Same colour and appearance throughout.
Stratified	- Consisting of a sequence of layers which are generally of contrasting texture or colour.
Laminated	- Stratified with layer thickness between 2 – 10 mm.
Thinly Laminated	- Stratified with layer thickness less than 2 mm.
Bedded	- Stratified with layer thickness greater than 10 mm.
Very Thinly Bedded (Flaggy)	- Stratified with layer thickness between 10 – 50 mm.
Thinly Beddy (Slabby)	- Stratified with layer thickness between 50 – 600 mm.
Thickly Beddy (Blocky)	- Stratified with layer thickness between 600 – 1200 mm.
Thick-Bedded (Massive)	- Stratified with layer thickness greater than 1200 mm.
Lensed	- Inclusions of small pockets of different soil, such as small lenses of sand material throughout a mass of clay.

Bedding Structures

Cross-bedding	- Internal ‘bedding’ inclined to the general bedding plane.
Ripple-bedding	- Internal ‘wavy bedding’.
Graded-bedding	- Internal gradation of grain size from coarse at base to finer at top of bed.
Horizontal bedded	- Internal bedding is parallel and flat lying.

Secondary Soil Structure (Post-Depositional)

Accretionary Structures

Includes nodules, concretions, crystal aggregates, veinlets, color banding, and:

Cementation	- Chemically precipitated material, commonly calcite ($CaCO_3$), binds the grains of soil, usually sandstone. Described as weak, moderate, or strong (ASTM D2488-00).
Salt Crystals	- Groundwater flowing though the soil/rock often precipitates visible amounts of salts. Calcite ($CaCO_3$), glauber salts ($Na_2Ca(SO_4)_2$), and gypsum ($CaSO_4*2H_2O$) are common.

Fracture Structures

Fracture	- A break or discontinuity in the soil or rock mass caused by stress exceeding the materials strength.
Joint	- A fracture along which no displacement has occurred.
Fissure	- A gapped fracture, which may open and close seasonally. Usually an extensive network of closely spaced fractures, giving the soil a ‘nuggetty’ structure.
Slickensides	- Fractures in clay that are slick and glossy in appearance, caused by shear movements.
Brecciated	- Contains randomly orientated angular fragments of a finer mass, usually associated with shear displacement in soils.
Fault	- A fracture or fracture zone along with displacement has occurred.
Blocky	- A cohesive soil that can be broken down into small angular lumps which resist further break down.

Symbols Used on Borehole Logs

Lithology Type

▤ Clay
▥ Silt
░ Sand
▒ Gravel
▓ Cobbles

▦ Till – oxidized
▦ Till – unoxidized
▒ Peat
▒ Fill (undifferentiated)

▓ Coal
▥ Topsoil or Organic Soil
▒ Concrete
■ Asphalt

▤ Clay Shale
▤ Sandstone
▧ Mudstone
▨ Bedrock (undifferentiated)

Borehole Completion and Backfill Materials

■ Bentonite
▒ Concrete
▦ Cover

▒ Cuttings
▒ Grout
░ Sand

▨ Slough
□ Solid Pipe
▤ Slotted Pipe

Soil Sample Type

■ Thin Walled Tube
▩ Driven Spoon

▤ Disturbed
▥ Core (any type)

▧ No Recovery

Groundwater Symbols

▼ Piezometric elevation as determined by a piezometer installation.

▽ Water levels measured in borings at time and under the conditions noted.

BOREHOLE LOG

Borehole: 101
Page: 1 of 2

Client:	RCMP	Northing:	5910526	Date Drilled:	16 January 2015
Project:	New Detachment	Easting:	514644	Drill:	MARL M10
Location:	Onion Lake, SK	Ground Elev.:	0	Drilling Method:	Solid Stem Auger
Project No.:	S2091	Top Casing Elev.:		Logged by:	JW

Depth (m): 0 – 10

Soil Description

TOPSOIL: 50mm Organic soil.

TILL, OXIDIZED: Clay, some fine to coarse-grained sand, some silt, trace gravel. Dark grayish brown (2.5Y 4/2). Calcareous. Moist. Stiff to hard. Homogeneous.

SAND: Fine to coarse-grained, silty, some clay. Light yellowish brown (2.5Y 6/4). Oxidized. Calcareous. Moist. Homogeneous.

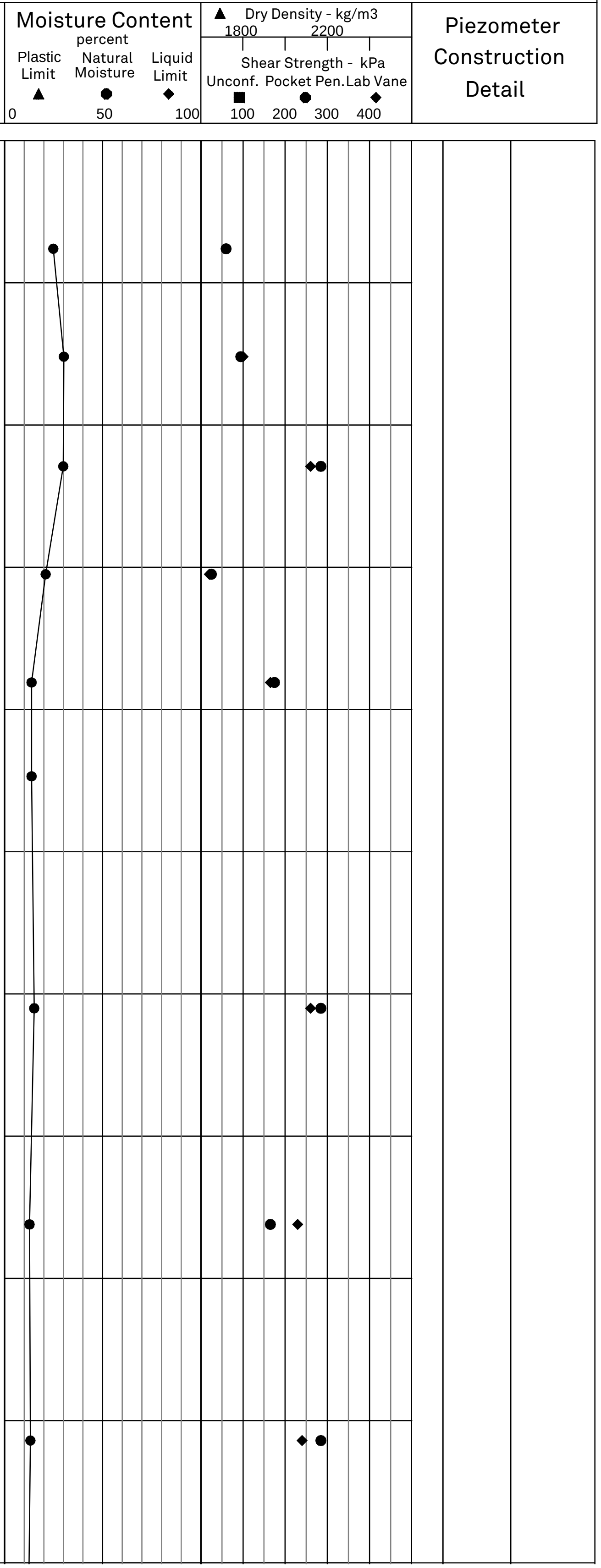
TILL, OXIDIZED: Clay, some fine to coarse-grained sand, some silt, trace gravel. Very dark grayish brown (2.5Y 3/2). Calcareous. Moist. Very stiff to very hard. Homogeneous.

@ 4.57m: Salt inclusions.

@ 6.10m: Iron staining.

TILL, UNOXIDIZED: Clay, some fine to coarse-grained sand, some silt, trace gravel. Very dark gray (2.5Y 3/1). Calcareous. Moist. Very stiff to very hard. Homogeneous.

Sample No.	SPT 'N'	USC	% Sulphate
JW01			
JW02			0.73
JW03			
JW04			
JW05			
JW06			
JW07			
JW08			
JW09			



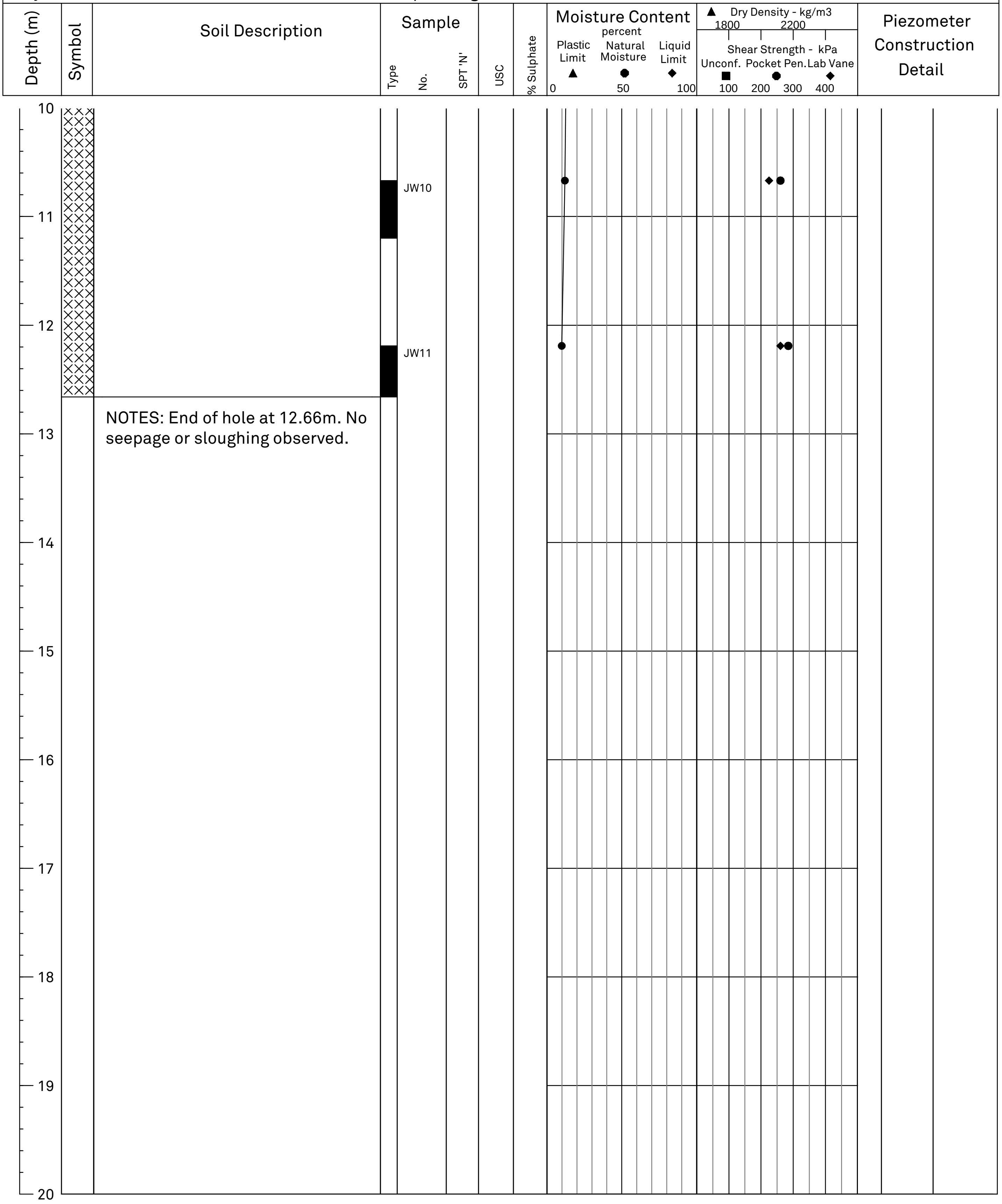
Geotech BH m Elev CAL v08.ldf

Clifton Associates

BOREHOLE LOG

Borehole: 101
Page: 2 of 2

Client:	RCMP	Northing:	5910526	Date Drilled:	16 January 2015
Project:	New Detachment	Easting:	514644	Drill:	MARL M10
Location:	Onion Lake, SK	Ground Elev.:	0	Drilling Method:	Solid Stem Auger
Project No.:	S2091	Top Casing Elev.:		Logged by:	JW



NOTES: End of hole at 12.66m. No seepage or sloughing observed.

Summary of Sampling and Laboratory Test Data

Sample				Water Content	Consistency				Gradation				Sulphate Content	Shear Strength			Dry Density
Depth	Number	Type	Recovery		Plastic Limit	Liquid Limit	Plasticity Index	USC	Gravel	Sand	Silt	Clay		Compression Test	Lab Vane	Pocket Penetrometer	
meters			mm	%	%	%	%		%	%	%	%	%	kPa	kPa	kPa	kg/m^3
0.30	JW35	Bag		28.4													1645
0.76	JW01	Sy	170	24.8											n/a	60	
1.52	JW02	Sy	240	30.2									0.73		100	95	
2.29	JW03	Sy	430	29.9											260	285	
3.05	JW04	Sy	500	21.0											20	25	
3.81	JW05	Sy	450	13.7											165	175	
4.57	JW06	Bag		13.8													
6.10	JW07	Sy	290	15.1											260	285	
7.62	JW08	Sy	430	12.7											230	165	
9.14	JW09	Sy	470	13.2											240	285	
10.67	JW10	Sy	530	12.0											225	260	
12.19	JW11	Sy	470	9.9											260	285	

Remarks

Approved by

Clifton Associates

Project No. S2091
Client RCMP
Project New Detachment Onion Lake Geotech
Location Onion Lake, SK
Borehole No. 101

Geotech BH m Elev CAL v08.ldf

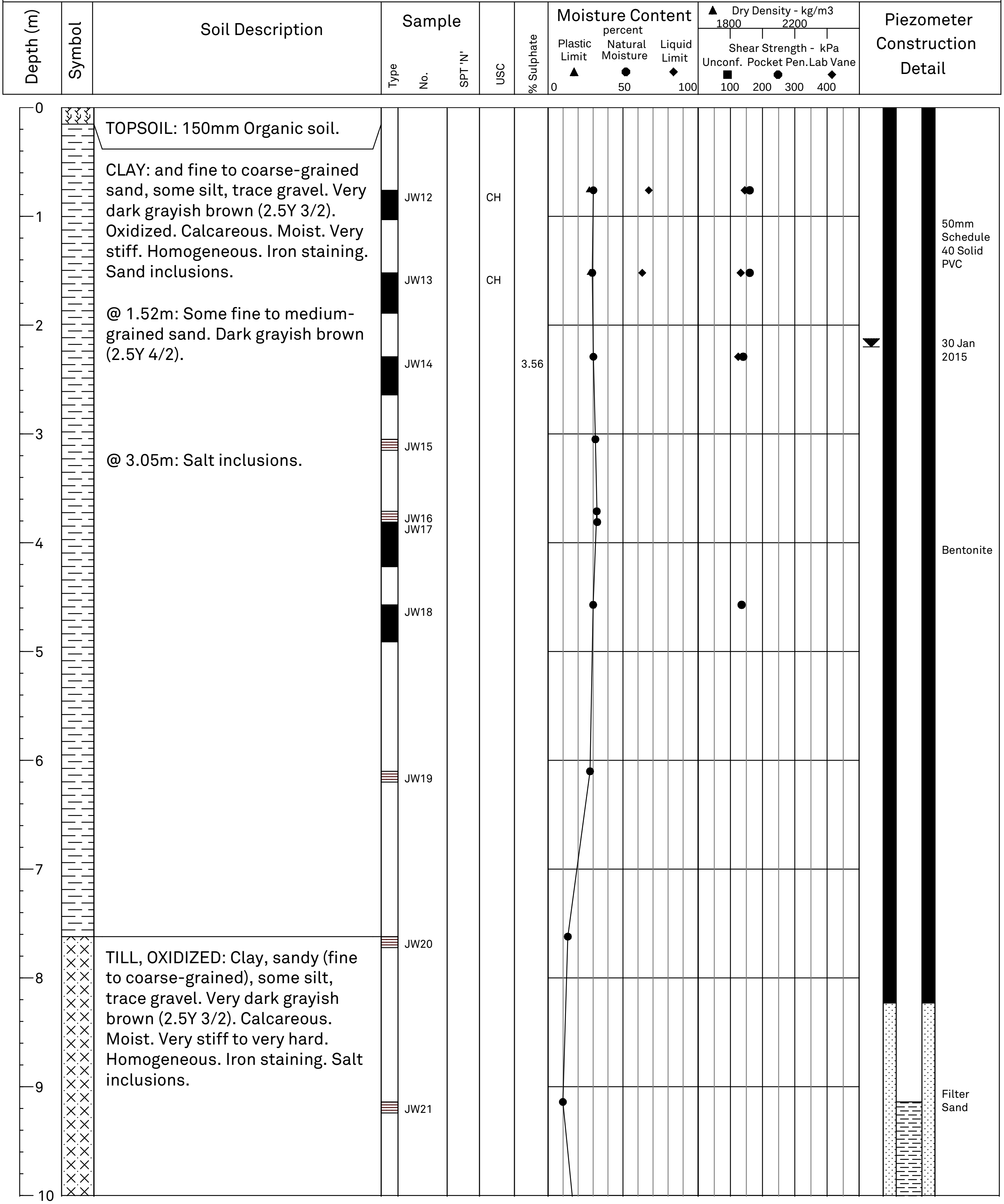
Clifton Associates

BOREHOLE LOG

Borehole: 102

Page: 1 of 2

Client:	RCMP	Northing:	5952632	Date Drilled:	16 January 2015
Project:	New Detachment	Easting:	566309	Drill:	MARL M10
Location:	Onion Lake, SK	Ground Elev.:	0	Drilling Method:	Solid Stem Auger
Project No.:	S2091	Top Casing Elev.:		Logged by:	JW



TOPSOIL: 150mm Organic soil.

CLAY: and fine to coarse-grained sand, some silt, trace gravel. Very dark grayish brown (2.5Y 3/2). Oxidized. Calcareous. Moist. Very stiff. Homogeneous. Iron staining. Sand inclusions.

@ 1.52m: Some fine to medium-grained sand. Dark grayish brown (2.5Y 4/2).

@ 3.05m: Salt inclusions.

TILL, OXIDIZED: Clay, sandy (fine to coarse-grained), some silt, trace gravel. Very dark grayish brown (2.5Y 3/2). Calcareous. Moist. Very stiff to very hard. Homogeneous. Iron staining. Salt inclusions.

No.	SPT 'N'	USC	% Sulphate
JW12		CH	
JW13		CH	
JW14			3.56
JW15			
JW16			
JW17			
JW18			
JW19			
JW20			
JW21			

Geotech BH m Elev CAL v08.ldf

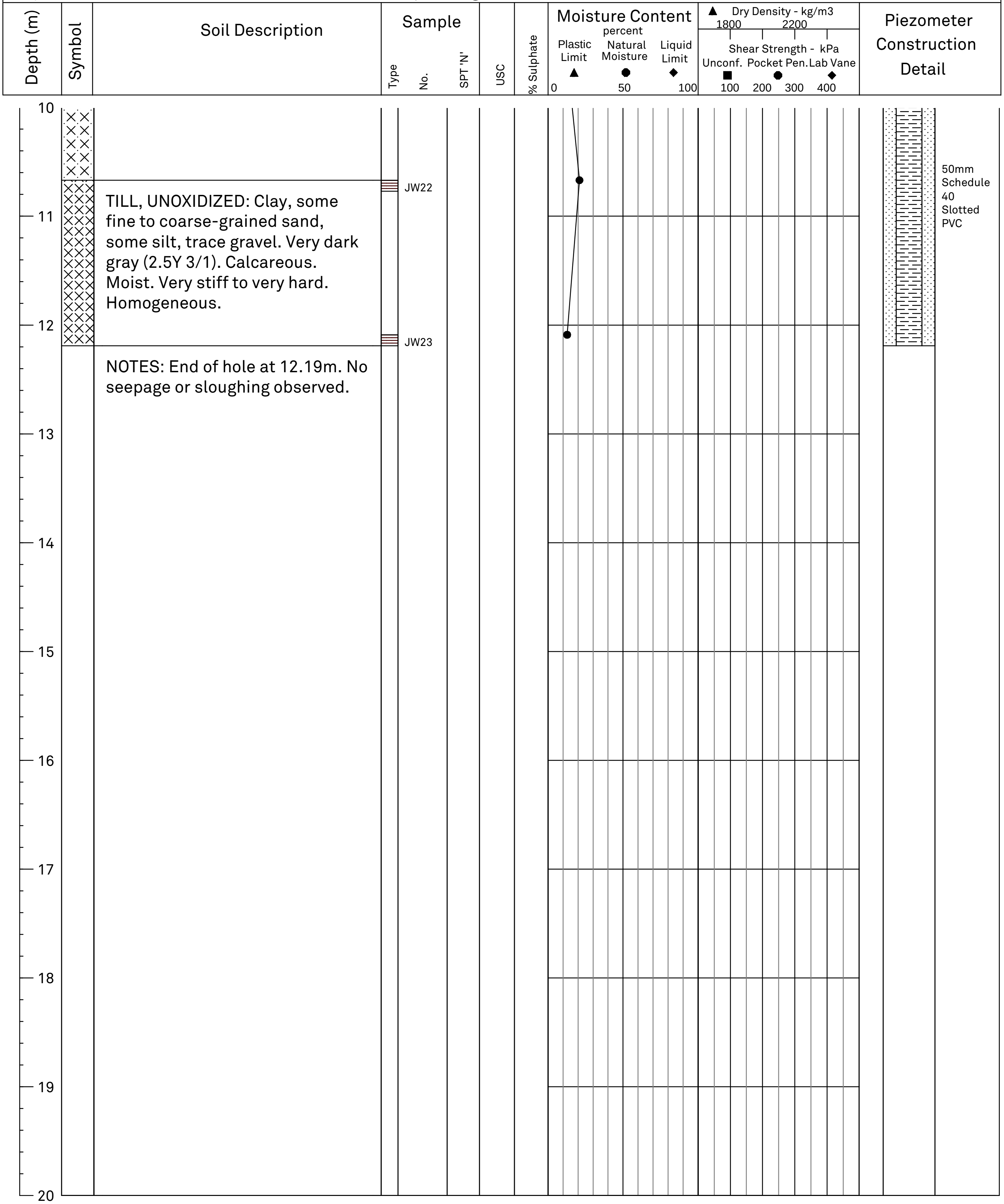
Clifton Associates

BOREHOLE LOG

Borehole: 102

Page: 2 of 2

Client:	RCMP	Northing:	5952632	Date Drilled:	16 January 2015
Project:	New Detachment	Easting:	566309	Drill:	MARL M10
Location:	Onion Lake, SK	Ground Elev.:	0	Drilling Method:	Solid Stem Auger
Project No.:	S2091	Top Casing Elev.:		Logged by:	JW



TILL, UNOXIDIZED: Clay, some fine to coarse-grained sand, some silt, trace gravel. Very dark gray (2.5Y 3/1). Calcareous. Moist. Very stiff to very hard. Homogeneous.

NOTES: End of hole at 12.19m. No seepage or sloughing observed.

Summary of Sampling and Laboratory Test Data

Sample				Water Content	Consistency				Gradation				Sulphate Content	Shear Strength			Dry Density
Depth	Number	Type	Recovery		Plastic Limit	Liquid Limit	Plasticity Index	USC	Gravel	Sand	Silt	Clay		Compression Test	Lab Vane	Pocket Penetrometer	
meters			mm	%	%	%	%		%	%	%	%	%	kPa	kPa	kPa	kg/m^3
0.30	JW36	Bag		26.9													1600
0.76	JW12	Sy	270	30.2	27.5	67.1	39.6	CH	0.5	39.2	60.3				145	160	
1.52	JW13	Sy	370	29.4	28.1	62.8	34.7	CH	0.0	18.3	81.7				132	160	
2.29	JW14	Sy	350	30.2									3.56		125	140	
3.05	JW15	Bag		31.6													
3.81	JW16	Sy	410	32.8											n/a	n/a	
3.81	JW17	Bag		32.4													
4.57	JW18	Sy	340	30.1											n/a	135	
6.10	JW19	Bag		27.9													
7.62	JW20	Bag		13.2													
9.14	JW21	Bag		9.8													
10.67	JW22	Bag		21.0													
12.19	JW23	Bag		12.7													

Remarks

Approved by

Clifton Associates

Project No. S2091

Client RCMP

Project New Detachment Onion Lake Geotech

Location Onion Lake, SK

Borehole No. 102

Geotech BH m Elev CAL v08.ldf

Clifton Associates

BOREHOLE LOG

Borehole: 103
Page: 1 of 2

Client:	RCMP	Northing:	5952611	Date Drilled:	16 January 2015
Project:	New Detachment	Easting:	566354	Drill:	MARL M10
Location:	Onion Lake, SK	Ground Elev.:	0	Drilling Method:	Solid Stem Auger
Project No.:	S2091	Top Casing Elev.:		Logged by:	JW

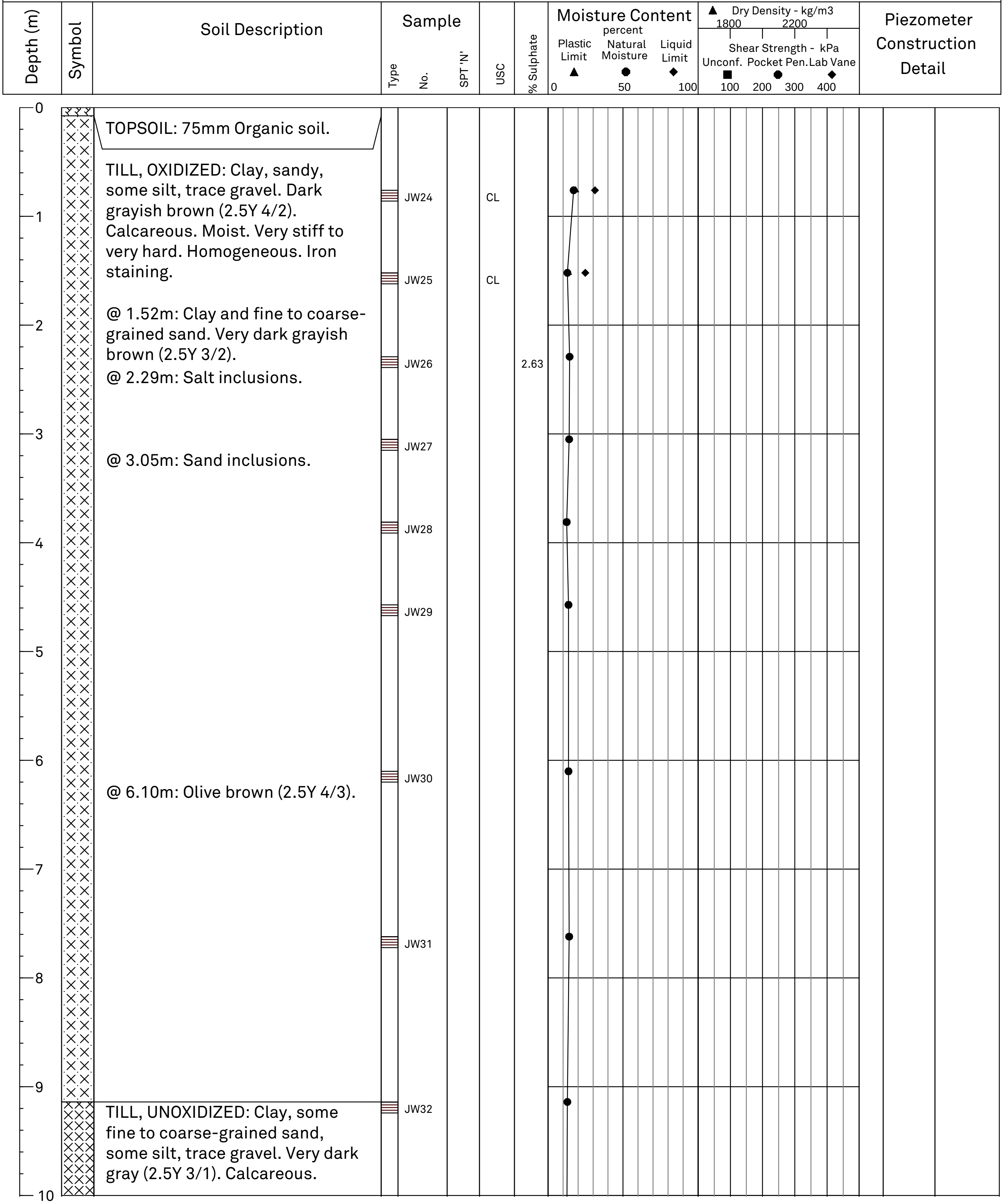
Depth (m)	Symbol	Soil Description
0		TOPSOIL: 75mm Organic soil.
		TILL, OXIDIZED: Clay, sandy, some silt, trace gravel. Dark grayish brown (2.5Y 4/2). Calcareous. Moist. Very stiff to very hard. Homogeneous. Iron staining. @ 1.52m: Clay and fine to coarse-grained sand. Very dark grayish brown (2.5Y 3/2). @ 2.29m: Salt inclusions. @ 3.05m: Sand inclusions. @ 6.10m: Olive brown (2.5Y 4/3).
		TILL, UNOXIDIZED: Clay, some fine to coarse-grained sand, some silt, trace gravel. Very dark gray (2.5Y 3/1). Calcareous.
10		

Sample Type	Sample No.	SPT 'N'	USC	% Sulphate
▤	JW24		CL	
▤	JW25		CL	
▤	JW26			2.63
▤	JW27			
▤	JW28			
▤	JW29			
▤	JW30			
▤	JW31			
▤	JW32			

Moisture Content percent
Plastic Limit ▲ Natural Moisture ● Liquid Limit ◆

▲ Dry Density - kg/m3
Shear Strength - kPa
Unconf. ■ Pocket Pen. ● Lab Vane ◆

Piezometer Construction Detail



Geotech BH m Elev CAL v08.ldf

Clifton Associates

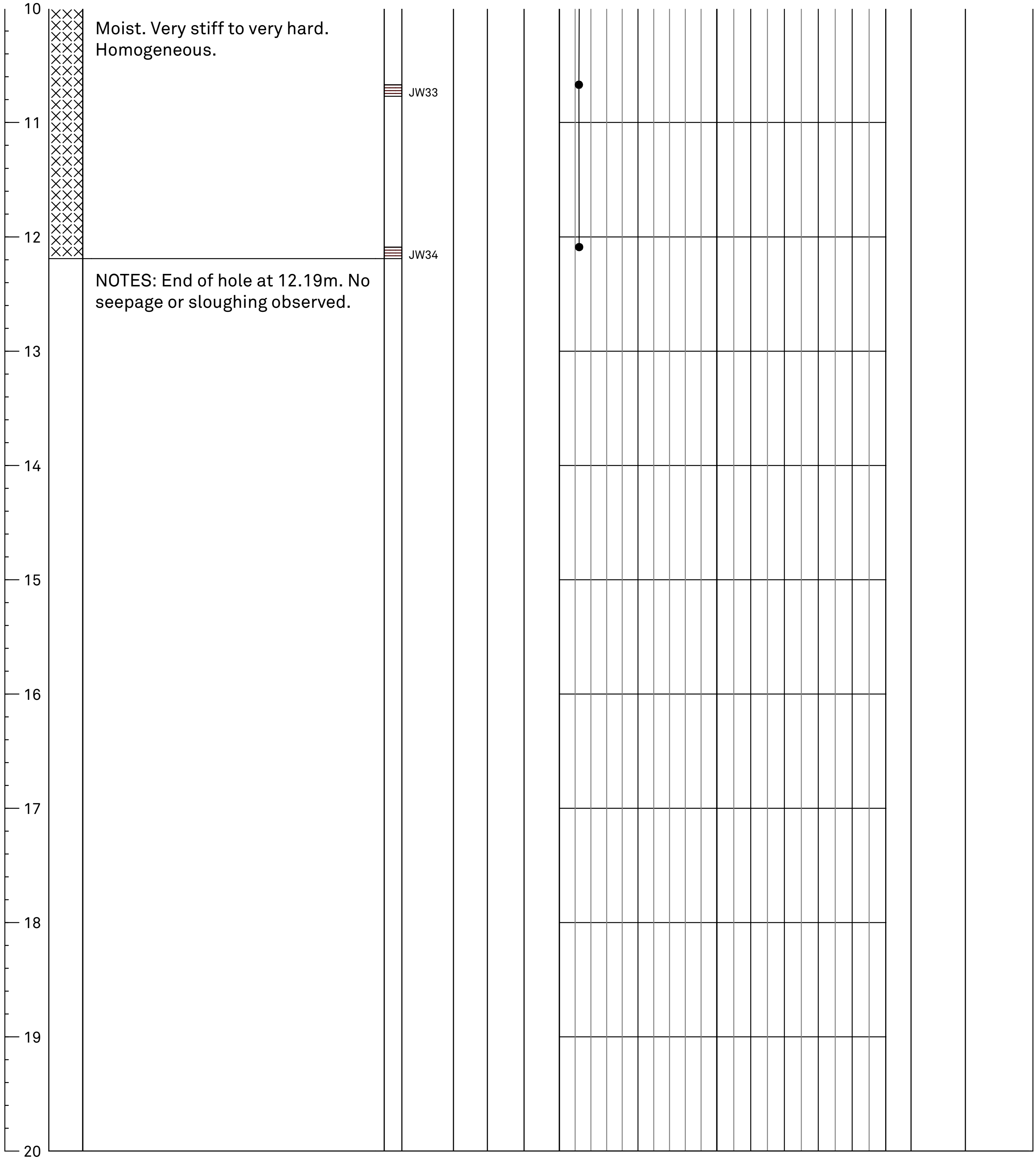
BOREHOLE LOG

Borehole: 103

Page: 2 of 2

Client:	RCMP	Northing:	5952611	Date Drilled:	16 January 2015
Project:	New Detachment	Easting:	566354	Drill:	MARL M10
Location:	Onion Lake, SK	Ground Elev.:	0	Drilling Method:	Solid Stem Auger
Project No.:	S2091	Top Casing Elev.:		Logged by:	JW

Depth (m)	Symbol	Soil Description	Sample Type	Sample No.	SPT 'N'	USC	% Sulphate	Moisture Content percent (Plastic Limit ▲, Natural Moisture ●, Liquid Limit ◆; 0, 50, 100)	▲ Dry Density - kg/m3 (1800, 2200); Shear Strength - kPa (Unconf. ■, Pocket Pen. ●, Lab Vane ◆; 100, 200, 300, 400)	Piezometer Construction Detail
10	☒	Moist. Very stiff to very hard. Homogeneous.	▤	JW33						
12	☒		▤	JW34						
12.19		NOTES: End of hole at 12.19m. No seepage or sloughing observed.								



Summary of Sampling and Laboratory Test Data

Sample				Water Content	Consistency				Gradation				Sulphate Content	Shear Strength			Dry Density
Depth	Number	Type	Recovery		Plastic Limit	Liquid Limit	Plasticity Index	USC	Gravel	Sand	Silt	Clay		Compression Test	Lab Vane	Pocket Penetrometer	
meters			mm	%	%	%	%		%	%	%	%	%	kPa	kPa	kPa	kg/m^3
0.76	JW24	Bag		17.0	18.4	31.3	12.9	CL	0.0	25.3	74.7						
1.52	JW25	Bag		12.9	13.6	24.8	11.2	CL	1.9	46.8	51.3						
2.29	JW26	Bag		14.3									2.63				
3.05	JW27	Bag		14.1													
3.81	JW28	Bag		12.4													
4.57	JW29	Bag		13.6													
6.10	JW30	Bag		13.6													
7.62	JW31	Bag		14.0													
9.14	JW32	Bag		12.8													
10.67	JW33	Bag		12.4													
12.19	JW34	Bag		12.6													

Remarks

Approved by

Clifton Associates

Project No.	S2091
Client	RCMP
Project	New Detachment Onion Lake Geotech
Location	Onion Lake, SK
Borehole No.	103

Appendix C

Clifton Associates

Recommended Specifications for Granular Materials

Clifton Associates

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340 Maxwell Crescent
Regina, SK S4N 5Y5

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Recommended Specifications for Granular Materials

1. Granular materials shall be composed of fragments of durable rock free from undesirable quantities of soft or flaky particles, topsoil, organic matter, clay or silt lumps, lumps of frozen granular soil, ice, snow or construction rubble.

2. The Pit Run Fill shall have a plasticity index less than 10 percent. The Crushed Base Course shall have a plasticity index less than 6 percent.

3. For Pit Run Sand, $\frac{D_{60}}{D_{10}}>6$, and $1<\frac{(D_{30})^2}{D_{10}xD_{60}}<3$. For Pit Run Gravel, $\frac{D_{60}}{D_{10}}>4$, and $1<\frac{(D_{30})^2}{D_{10}xD_{60}}<3$.

4. Granular materials shall be excavated, loaded, hauled, placed and levelled in such a manner to prevent contamination with undesirable materials described in Point 1 above and to prevent excessive segregation of coarse and fine particles.

5. Granular material shall conform to the following gradation specifications:

Percent by Weight Passing U.S. Standard Sieve Series							
Sieve	Pit Run Gravel Fill	Pit Run Sand Fill	Crushed Base Course				
			32	33	34	35	36
50.0 mm	100						
25.0 mm	85 – 100		100				
18.0 mm	80 – 100		87 – 100	100	100	100	100
12.5 mm	70 – 100	100	79 – 93	81 – 100	91 - 100	81 - 100	91 – 100
5.0 mm	50 – 85	75 – 100	47 – 77	50 – 80	70 - 85	50 - 85	70 – 85
2.0 mm	35 – 75	50 – 90	29 – 56	32 – 52	45 - 65	32 - 65	45 – 70
900 µm	25 – 50	30 – 75	18 – 39	20 – 35	28 - 43	20 - 43	28 – 51
400 µm	15-35	15 – 50	13 – 26	15 – 25	20 - 30	15 - 30	20 – 35
160 µm	8 – 22	5 – 30	7 – 16	8 – 15	11 - 18	8 - 18	11 – 21
71 µm	0 - 13	0 – 15	6 - 11	7 – 10	8 - 12	7 - 12	8 – 13

Appendix D

Clifton Associates
pH Test Results

Clifton Associates

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Clifton Associates Ltd.
ATTN: Geoff Haanen
4 - 1925 1st Ave N
Saskatoon SK S7K 6W1

Date Received:	05-FEB-15
Report Date:	11-FEB-15 09:57 (MT)
Version:	FINAL

Client Phone: 306-975-0401

Certificate of Analysis

Lab Work Order #:	**L1574707**
Project P.O. #:	NOT SUBMITTED
Job Reference:	S2091
C of C Numbers:	
Legal Site Desc:	

B. Morgan

Brian Morgan
Account Manager

ALS ENVIRONMENTAL ANALYTICAL REPORT

Sample Details/Parameters	Result	Qualifier*	D.L.	Units	Extracted	Analyzed	Batch
L1574707-1 JW02 Sampled By: JW on 15-JAN-15 Matrix:							
pH and EC (Saturated Paste)							
% Saturation	71.5		1.0	%	09-FEB-15	10-FEB-15	R3145409
pH in Saturated Paste	7.66		0.10	pH	09-FEB-15	10-FEB-15	R3145409
Conductivity Sat. Paste	6.94		0.10	dS m-1	09-FEB-15	10-FEB-15	R3145409
L1574707-2 JW14 Sampled By: JW on 15-JAN-15 Matrix:							
pH and EC (Saturated Paste)							
% Saturation	98.3		1.0	%	09-FEB-15	10-FEB-15	R3145409
pH in Saturated Paste	7.33		0.10	pH	09-FEB-15	10-FEB-15	R3145409
Conductivity Sat. Paste	6.23		0.10	dS m-1	09-FEB-15	10-FEB-15	R3145409
L1574707-3 JW26 Sampled By: JW on 15-JAN-15 Matrix:							
pH and EC (Saturated Paste)							
% Saturation	54.0		1.0	%	09-FEB-15	10-FEB-15	R3145409
pH in Saturated Paste	7.57		0.10	pH	09-FEB-15	10-FEB-15	R3145409
Conductivity Sat. Paste	6.63		0.10	dS m-1	09-FEB-15	10-FEB-15	R3145409

* Refer to Referenced Information for Qualifiers (if any) and Methodology.

Reference Information

Test Method References:

ALS Test Code	Matrix	Test Description	Method Reference**
SAT/PH/EC-SK	Soil	pH and EC (Saturated Paste)	CSSS 18.2.2/CSSC 3.14/CSSS 18.3.1

pH of a saturated soil paste is measured using a pH meter. After equilibration, an extract is obtained by vacuum filtration with conductivity of the extract measured by a conductivity meter.

** ALS test methods may incorporate modifications from specified reference methods to improve performance.

The last two letters of the above test code(s) indicate the laboratory that performed analytical analysis for that test. Refer to the list below:

Laboratory Definition Code	Laboratory Location
SK	ALS ENVIRONMENTAL - SASKATOON, SASKATCHEWAN, CANADA

Chain of Custody Numbers:

GLOSSARY OF REPORT TERMS

Surrogates are compounds that are similar in behaviour to target analyte(s), but that do not normally occur in environmental samples. For applicable tests, surrogates are added to samples prior to analysis as a check on recovery. In reports that display the D.L. column, laboratory objectives for surrogates are listed there.
mg/kg - milligrams per kilogram based on dry weight of sample
mg/kg wwt - milligrams per kilogram based on wet weight of sample
mg/kg lwt - milligrams per kilogram based on lipid-adjusted weight
mg/L - unit of concentration based on volume, parts per million.
< - Less than.
D.L. - The reporting limit.
N/A - Result not available. Refer to qualifier code and definition for explanation.

Test results reported relate only to the samples as received by the laboratory.
UNLESS OTHERWISE STATED, ALL SAMPLES WERE RECEIVED IN ACCEPTABLE CONDITION.
Analytical results in unsigned test reports with the DRAFT watermark are subject to change, pending final QC review.

L1574707-COFC

ain of Custody (COC) / Analytical Request Form

Canada Toll Free: 1 800 668 9878

Affix ALS barcode label here
(lab use only)

COC Number: 14 - 420195

Page 1 of 1

Report To
Company: Clifton
Contact: Geoff Hannon
Address: 4-1925 1st Ave N
Phone:

Report Format / Distribution
Select Report Format: ☑ PDF ☐ EXCEL ☐ EDD (DIGITAL)
Quality Control (QC) Report with Report ☐ Yes ☐ No
☐ Criteria on Report - provide details below if box checked
Select Distribution: ☑ EMAIL ☐ MAIL ☐ FAX
Email 1 or Fax Jon_Osback@clifton.ca
Email 2

Select Service Level Below (Rush Turnaround Time (TAT) is not available for all tests)
R ☒ Regular (Standard TAT if received by 3pm)
P ☐ Priority (2-4 business days if received by 3pm)
E ☐ Emergency (1-2 business days if received by 3pm)
E2 ☐ Same day or weekend emergency if received by 10am – contact ALS for surcharge.
Specify Date Required for E2,E or P:

Invoice To Same as Report To ☒ Yes ☐ No
Copy of Invoice with Report ☒ Yes ☐ No
Company:
Contact:

Invoice Distribution
Select Invoice Distribution: ☒ EMAIL ☐ MAIL ☐ FAX
Email 1 or Fax Same as above
Email 2

Project Information
ALS Quote #:
Job #: S2091
PO / AFE:
LSD:

Oil and Gas Required Fields (client use)
Approver ID:
Cost Center:
GL Account:
Routing Code:
Activity Code:
Location:

ALS Lab Work Order # (lab use only)

ALS Contact: Brian Morgan
Sampler: JW

Analysis Request

Indicate Filtered (F), Preserved (P) or Filtered and Preserved (F/P) below

ALS Sample # (lab use only)	Sample Identification and/or Coordinates (This description will appear on the report)	Date (dd-mmm-yy)	Time (hh:mm)	Sample Type	Saturated Paste pH													Number of Containers
	JW02	15-Jan-15		G-B	X													
	JW14	↓		↓	X													
	JW26	↓		↓	X													

Drinking Water (DW) Samples[1] (client use)
Are samples taken from a Regulated DW System?
☐ Yes ☐ No
Are samples for human drinking water use?
☐ Yes ☐ No

Special Instructions / Specify Criteria to add on report (client Use)

SAMPLE CONDITION AS RECEIVED (lab use only)
Frozen ☐
Ice packs Yes ☐ No ☐
Cooling Initiated ☐
SIF Observations Yes ☐ No ☐
Custody seal intact Yes ☐ No ☐

INITIAL COOLER TEMPERATURES °C			FINAL COOLER TEMPERATURES °C		
14°					

SHIPMENT RELEASE (client use)
Released by:
Date: 05-Feb-15
Time: 0932

INITIAL SHIPMENT RECEPTION (lab use only)
Received by: NK
Date: 5 Feb 2015
Time:

FINAL SHIPMENT RECEPTION (lab use only)
Received by:
Date:
Time:

REFER TO BACK PAGE FOR ALS LOCATIONS AND SAMPLING INFORMATION

WHITE - LABORATORY COPY YELLOW - CLIENT COPY

NA-FM-0326e v08 Front/03 October 2013

Failure to complete all portions of this form may delay analysis. Please fill in this form LEGIBLY. By the use of this form the user acknowledges and agrees with the Terms and Conditions as specified on the back page of the white - report copy.

1. If any water samples are taken from a **Regulated Drinking Water (DW) System**, please submit using an **Authorized DW COC form**.

Clifton Associates

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