



January 11, 2019

## École J.H. Sissons School: Geotechnical Investigation Report

In July 2018 a contract was awarded to Wood Environment and Infrastructure Solutions to conduct a geotechnical evaluation of foundation soil conditions for the replacement of École J.H. Sissons School.

Fourteen boreholes were drilled in the parking lot and playground areas between August 13 and 17 2018. Soil samples were taken from the boreholes and sent to a laboratory for analysis.

The Geotechnical Investigation Report was submitted by Wood on September 24, and reviewed by the Department of Infrastructure. Wood was subsequently asked to provide an additional option for piles driven to refusal. On November 8 Wood provided a Memorandum containing recommendations for driven steel piles. The Geotechnical Investigation Report and subsequent Memorandum accompany this cover page.

The GNWT concluded that the existing school site, on the rock outcrop, would be the best location for the new school. This recommendation was based on the risks and potential long term cost implications of the soil conditions in the playground and parking lot, including:

- A high water table that would cause challenges during construction and require ongoing maintenance.
- The impracticality of rock-socketed steel piles because of problems with groundwater.
- The failure of the drilling program to locate bedrock in some locations.
- The need to keep the soil under and around the foundation from freezing in order to avoid frost jacking or settlement.

These risks can be avoided by building on the existing school site.



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8 November 2018  
YX18006

Government of the Northwest Territories  
P.O. Box 1320, 5009 – 49th Street  
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**Attention: Mr. Barry Ward**

Dear Sir,

**Re: Driven Steel Pile Recommendations  
École J.H. Sissons School Replacement  
Yellowknife, North West Territories**

## **1.0 INTRODUCTION**

Wood Environment & Infrastructure Solutions (Wood) completed a geotechnical evaluation for the subject project and presented the results in a report titled "Geotechnical Investigation Proposed School Replacement, 5700 51A Ave, Yellowknife, NT X1A 1G7, Wood File No.: YX18006", dated 24 September 2018. Wood was requested by Barry Ward of the Government of the Northwest Territories to update the geotechnical recommendations to include recommendations for driven steel piles.

## **2.0 DRIVEN STEEL PILES**

Relative to the proposed development, the subsurface soil and groundwater conditions observed in the test borings are considered only fair. Adequate soil cover, or insulation, will be required to maintain perimeter footings below the seasonal frost penetration depth. For driven steel piles, shallow refusal on bedrock is likely to occur before minimum pile depths of either 7 m for exterior piles and/or 5 m for interior piles are achieved at some locations and thus will require added insulation.

## **2.1 DESIGN FOR COMPRESSIVE LOADS**

The depth to bedrock, and variability of the depth to bedrock encountered would not allow for reliable pile design using skin friction, and thus should be limited to end-bearing.

For steel piles driven through the native dense sand onto bedrock, an end-bearing resistance may be included in the design for compressive loads. The unfactored ultimate end-bearing resistance at the pile toe,  $q_t$ , for piles driven to at least 20 pile diameters depth, may be taken as 12500 kPa, applied to the gross area at the pile tip (the area enclosed by the outside diameter of the pile).

To provide resistance against frost jacking forces, piles should be driven to a minimum depth of 7 m for exterior piles and 5 m for interior piles. The center-to-center pile spacing should be a minimum of 3 pile diameters. The working load on a steel pile should be limited to no more than the allowable fiber stress of the steel, which should be determined by multiplying the cross-sectional area of steel by  $0.35 f_y$ , where  $f_y$  is the yield strength of the steel. This recommendation is provided mainly to control driving stresses, as past



experience has indicated that if the compressive load capacities are reduced to this degree, the likelihood of structural damage caused by pile driving is also reduced.

## **2.2 INSTALLATION AND MONITORING OF DRIVEN STEEL PILES**

Steel piles should be driven using maximum hammer energies of 450 to 600 J per blow for each square centimeter of steel in the pile cross-section. To limit potential for structural damage to the pile, the piles should not be driven beyond practical refusal, which may be taken as 10 to 12 blows per 25 mm penetration for the last 250 mm of penetration, for this range of hammer energy. This criterion is a preliminary guide to estimate the size of pile driving hammer that would be required. The ability of a pile driving hammer to drive the proposed piles to the required capacity should be confirmed using wave equation analyses (*GRLWEAP* software) once details regarding the proposed hammer configuration and the pile size and wall thickness are known. The required termination criteria should also be determined using wave equation analyses for the given design loads.

Based on borehole data, there may be zones in the stratigraphy, or parts of the site where hard driving conditions will be encountered. Boulders and cobbles may also be encountered during pile driving. Typically, the hard driving conditions could be encountered in the very dense sand with SPT 'N' values greater than 40. Where hard driving is encountered, the steel pipe piles may need to be driven in pre-bored pilot holes. The diameter of the pre-bored pilot hole should be limited to less than 85 percent of the outside diameter of the pile, and the pile tip should be driven below the base of the pre-bored pilot holes. The use of pre-bored pilot holes for steel H-piles is not recommended since the pile section would not be in full contact with the surrounding soil, thus compromising the frictional resistance and the lateral load resistance.

It is recommended that piles within a group be driven from the center of the group outwards. Where end-bearing has been included in the design, or where the piles are driven using a termination criterion, the elevation of the tops of piles previously installed within seven pile diameters should be monitored as adjacent piles are driven in order to determine if heaving of the previously installed piles has occurred. Piles that have heaved must be re-driven to at least their initial embedment depths.

Prior to the pile installation, the piles should be inspected to confirm that the material specifications are satisfied. The piles should be free from protrusions, including protruding welds which could create voids in the soil around the pile during driving. If a driving shoe is used, it must not protrude beyond the outside diameter of the pipe pile, or beyond the exterior sides of the flanges in the case of the steel H-piles.

Monitoring of the pile installations by qualified personnel is recommended to verify that the piles are installed in accordance with design assumptions and that driving criteria are satisfied. For each pile, a complete driving record in terms of the number of blows per 250 mm of penetration should be recorded by the inspector and reviewed daily during pile installation by a qualified geotechnical engineer.

## **2.3 FROST DESIGN CONSIDERATION FOR PILES**

Piles which support unheated structures, or those piles along the exterior sides of heated facilities will be subject to potential frost heaving forces acting on the underside of attached grade beams or pile caps, and adfreezing pressures acting along the pile shafts. The potential for frost heaving forces can be greatly reduced by the placement of a compressible material or by providing a void between the underside of the pile cap or grade beam and the soil. A void-forming product is recommended. The minimum thickness of the void should be 100 mm. Should a compressible material be used, the uplift pressure acting on the undersides of the grade beams or pile caps may be taken as the crushing strength of the compressible medium.



The finished grade adjacent to each pile cap or grade beam should be capped with well-compacted clay and sloped away so that the surface runoff is not allowed to infiltrate and collect in the void space or in the compressible medium. If water is allowed to accumulate in the void space or the compressible medium becomes saturated, the beneficial effect will be negated, and frost heaving pressures will occur on the undersides of the pile caps or grade beams.

With respect to frost adfreeze stresses on the pile shafts, the recommended minimum depth of embedment of 7 m should be provided for perimeter piles. In the case of piles supporting relatively large downward dead loads, the pile embedment depth may be reduced. In such cases, the required embedment depth may be rationally determined. An adfreezing stress of 65 kPa acting along the pile shaft is recommended within the frozen zone which should be taken as 2.5 m for exterior piles supporting a heated structure, and 3.5 m for piles supporting an unheated structure. The forces resisting the adfreeze stress will include the dead weight acting on the pile, weight of the pile, and frictional resistance of the pile below the frozen zone.

## 2.4 PILE CAPS AND GRADE BEAMS

Precautions should be taken to reduce the potential for heaving of the pile caps and grade beams due to frost penetration. The potential for frost heaving forces can be greatly reduced by the placement of a compressible material or by providing a void between the underside of the pile cap and the soil. A product such as *Voidform* (or equivalent) is recommended. The minimum thickness of the void should be 100 mm. Should a compressible material be used as an alternative to *Voidform*, the uplift pressure acting on the underside of the pile caps may be taken as the crushing strength of the compressible medium. The finished grade adjacent to each pile cap should be capped with clay and sloped away so that surface runoff is not allowed to accumulate in the void space or in the compressible medium. If water is allowed to accumulate in the void spaces, the beneficial effect of the void space will be negated and frost-heaving pressures acting on the underside of the pile caps will occur.

Adfreeze stresses along the sides of pile caps and buried substructures can be reduced by the installation of a "bond-break" within the zone of frost penetration. For grade beams, pile caps and most substructures, a suitable bond-break medium could consist of a *Dow Ethafoam* product. A smooth geosynthetic liner material, fixed to the shaft of the pile or to the sides of the pile cap would also be a suitable bond-break.



**3.0 CLOSURE**

Wood trusts that this information will meet your present requirements. Should you have any questions or concerns, please do not hesitate to contact the office.

Respectfully submitted,

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a division of Wood Canada Limited**

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## **Geotechnical Investigation**

Proposed School Replacement  
École J.H. Sissons School  
5700 51a Ave, Yellowknife, NT X1A 1G7  
Project # YX18006

Prepared for:

**Government of the Northwest Territories**

P.O. Box 1320, 5009 – 49th Street, Yellowknife, NT X1A 2L9

24-Sep-18

## **Geotechnical Investigation**

### **Proposed School Replacement**

### **École J.H. Sissons School**

**Project # YX18006**

**Prepared for:**

Government of the Northwest Territories  
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## 1.0 Introduction

### 1.1 General

Wood Environment & Infrastructure Solutions (Wood) was retained by the Government of the Northwest Territories (GNWT) to conduct a geotechnical evaluation of the foundation soil conditions for the proposed École J.H. Sissons School replacement in Yellowknife, North West Territories (Lot 17, Block 143, Plan 58508). The purpose of the evaluation was to provide geotechnical engineering recommendations for subgrade preparation, foundation design and geotechnically related aspects for the proposed structures.

This report summarizes the results of the field and laboratory work and provides discussion and recommendations for the design and construction of foundation systems, slabs-on-grade, installation of site services, site grading, pavement, backfilling procedures, and cement type for subsurface concrete.

Authorization to proceed with the scope of work, as defined in Wood's proposal YX18P016, was received from the GNWT, dated 26 July 2018.

### 1.2 Site and Project Description

The proposed school is proposed to be located on the existing school's lot, located at 5700 51a Ave in Yellowknife, NWT on the lower portion of the site such that the existing school can remain open during its construction. The proposed school will likely consist of a two-storey structure, approximately 3,800 m<sup>2</sup> in area (total), potentially with a basement or crawlspace. Design consideration will be given to preserving the existing trees and playground areas of the site. Asphalt parking and accessways are expected to encompass the majority of the remaining areas on site.

The site topography is generally flat lying with two distinct elevations. The current parking lot and the bus loading zones sits approximately 3 m to 4 m above the soccer field, baseball diamond and lower level playground.

A site plan showing the location of boreholes advanced during this investigation is shown on Figure 1 in Appendix A.

## 2.0 Geotechnical Investigation

Prior to borehole drilling, Wood engaged the services of White Bear Geomatics Limited (White Bear) to identify underground services and utilities at the borehole locations. All borehole locations were cleared of the above ground and underground utilities before drilling commenced.

From 13 through 17 August 2018, fourteen (14) boreholes (BH18-01 to BH18-14) were advanced at the site. Nine (9) boreholes (BH18-01 to BH18-09) were positioned throughout the soccer field, baseball diamond and playground areas and were advanced to depths ranging from 3.8 m to 12.0 m below existing grade. Five (5) boreholes (BH18-10 to BH18-14) were positioned throughout the existing parking lot and the bus loading zone and were advanced to depths ranging from 2.4 m to 13.5 m below existing grade.

Borehole locations were surveyed by White Bear personnel using a Trimble R10 RTK base and rover with a horizontal accuracy of  $\pm 0.008$  m and a vertical accuracy of  $\pm 0.015$  m. The GPS coordinates were referenced to NAD 83 CSRS (2010.0), UTM Zone 11. The borehole GPS coordinates and elevations are noted on the borehole logs.

The boreholes were advanced using a track-mounted drill rig using continuous flight, 150 mm diameter solid-stem augers (SSA) and continuous flight, 200 mm hollow-stem augers (HSA). Supervision of drilling,

soil sampling, and logging of the soil strata was performed by Wood geotechnical personnel. Detailed borehole logs summarizing the sampling, field testing groundwater and subsurface conditions encountered at the borehole locations are presented in Appendix A.

The soil conditions encountered during drilling were described in accordance with the Modified Unified Soil Classification System (MUSCS) as per the Explanation of Terms & Symbols in Appendix A. Soil sampling and evaluation of in-situ soil consistency and relative density consisted of the following:

- Disturbed auger samples were obtained at depth intervals varying from 0.3 m to 1.5 m for moisture content determinations (labeled G#). The moisture content profiles are shown on the borehole logs.
- Standard Penetration Tests (SPT's) were conducted in approximately half of the boreholes at 1.5 m depth intervals to evaluate the consistency of the various soil strata. SPT results, defined as the number of blows required to drive the standard SPT split-spoon sampler 300 mm into the soil, were recorded and are noted on the borehole logs as the SPT 'N' values.
- Pocket penetrometer (PP) readings were taken on disturbed soil samples to aid in determining the relative consistency of the cohesive soils.

The depth to slough (collapsed soil) and groundwater in all boreholes were measured upon drilling completion. A 50-mm diameter PVC standpipe was installed in boreholes BH18-04, BH18-12 and BH18-14 for short term monitoring of the current groundwater levels. The annulus of the standpipe boreholes, including the slotted sections, were backfilled with drill cuttings up to the slotted length of the standpipe, and a 2 m thick bentonite cap was placed at ground surface. The remaining boreholes were backfilled with a combination of auger cuttings and a surficial bentonite cap.

The water levels in the standpipes were measured by Wood, approximately 11 days after drilling completion, on 28 August 2018.

Following completion of the field drilling program, a laboratory testing program was conducted on selected soil samples and consisted of: moisture content determinations, water soluble sulphate tests and grain size analysis. The results of the laboratory program are noted on the borehole logs.

## **3.0 Subsurface Soil Conditions**

### **3.1 General Stratigraphy**

The generalized stratigraphy encountered at the borehole locations consisted of gravel and sand fill, underlain in descending order by sand and silty sand or silt. Detailed descriptions of the soil conditions encountered in the boreholes are provided on the borehole logs (BH18-01 to BH18-14) in Appendix A.

For discussion purposes, a general description of soil types encountered at the borehole locations is presented in the succeeding subsections.

#### **3.1.1 Gravel Fill**

Blast rock gravel fill was encountered from surface at boreholes BH18-10 and BH18-13 and extended to depths varying between 0.4 m and 0.5 m below existing grade. Surficial blast rock gravel was encountered in BH18-11, BH18-12 and BH18-14 where it was used as surfacing material. It was observed to be generally sandy, and was coarse grained, well graded, brown to dark brown and moist.

#### **3.1.2 Sand Fill**

Sand fill was encountered from surface at boreholes BH18-01, BH18-02, BH18-05, BH18-08, BH18-11 and BH18-14 and extended to depths varying between 0.1 m to 1.6 m below existing grade. It was observed to be generally silty with trace clay, and was fine to medium grained, well graded, brown and damp to wet. Properties measured in the sand fill were:

- Moisture Content:
  - Varied between 3 and 17 percent, with the majority of values ranging between 14 and 17 percent.
- SPT 'N' Values:
  - One (1) SPT value at 14, indicating a compact relative density.

### 3.1.3 Sand

Sand was encountered below either the gravel fill or sand fill at BH18-01, BH18-02, BH18-05, BH18-08, BH18-10, BH18-11 and BH18-13 and was encountered at surface in all other boreholes. The sand extended to either the termination depth on inferred bedrock (termination depths of 2.4 m to 9.8 m below existing grade) or to some depth beyond the termination depth (2.4 m to 9.8 m below existing grade). At BH18-01 the sand extended to a depth of 8.0 m below existing grade. It was observed to be generally silty to trace silt with trace amounts of clay, and was fine to medium grained, well graded, dense to very dense, brown to greyish brown in colour and very moist to wet. It was observed that the silt content and relative density increased with depth. Properties measured in the sand were:

- Moisture Content:
  - Varied between 6 and 21 percent with the majority of the values ranging between 12 and 19 percent, indicating that the sand was generally wet.
- SPT 'N' Values:
  - Generally above 50, indicating a very dense relative density with four (1) values indicating a dense relative density and 11 values indicating a compact relative density. SPT 'N' values generally increased with depth.
- Five (5) particle size distribution analyses conducted on samples of sand yielded particle size distributions of:
  - Gravel: 0 to 1 percent
  - Sand: 47 to 96 percent
  - Clay and Silt: 4 to 53 percent

### 3.1.4 Silt

A silt layer was encountered in the sand at BH18-01 from 6.5 m to 6.8 m, and silt was present at BH18-14 below the sand and extended to the termination depth of 7.1 m on inferred bedrock. It was observed to generally have some sand, trace clay, trace gravel; and was low plastic, stiff to very stiff, grey to greyish brown in colour and moist to very moist. Properties measured in the silt were:

- Moisture Content:
  - Varied between 11 and 18 percent.
- SPT 'N' Values:
  - Three values of over 50, indicating a hard consistency and two values indicating a stiff to very stiff consistency.
- One grain size analyses conducted on a sample of silt yielded grain size distributions of:
  - Gravel: 0%
  - Sand: 28%



- Silt and Clay: 72%

### 3.2 Groundwater and Sloughing Conditions

Accumulations of collapsed soils (slough) and groundwater levels were measured approximately ten minutes following drilling completion at each of the borehole locations. Due to the sloughing and sub-surface ground water conditions, hollow stem augers were utilized in half of the boreholes to prevent the borehole from sloughing in. Moderate to heavy seepage and sloughing were observed in the all the boreholes, with the majority of the boreholes encountering heavy seepage and sloughing conditions. Groundwater levels in the standpipes were measured approximately 11 days following drilling. Measured slough and groundwater levels are summarized in **Table 1**.

**Table 1: Measured Slough and Groundwater Levels**

Borehole	Termination Depth (m)	Depth to Top of Slough at Drilling Completion (m)	Groundwater Level at Drilling Completion (m)	Groundwater Level on 28 August 2018 (m)	Well Screen Interval (m)
BH18-01	8.0	1.1	0.9	No standpipe	---
BH18-02	12.0	6.1	0.9	No standpipe	---
BH18-03	11.7	1.4	0.2	No standpipe	---
BH18-04	6.6	1.2 <sup>1</sup>	0.3 <sup>1</sup>	1.5	1.5 to 4.5
BH18-05	10.5	0.6	0.6	No standpipe	---
BH18-06	8.8	0.2	0.1	No standpipe	---
BH18-07	5.7	0.4	0.4	No standpipe	---
BH18-08	3.8	0.2	0.2	No standpipe	---
BH18-09	5.5	0.6	0.6	No standpipe	---
BH18-10	2.4	1.8	1.8	No standpipe	---
BH18-11	9.8	1.3	1.2	No standpipe	---
BH18-12	9.8	0.9 <sup>1</sup>	0.9 <sup>1</sup>	1.5	1.5 to 4.5
BH18-13	13.5	0.6	0.6	No standpipe	---
BH18-14	7.1	0.6 <sup>1</sup>	0.6 <sup>1</sup>	See note 2	1.5 to 4.5

Notes:

1. Depth measurement after the solid stem augers were switched out for hollow stem augers.
2. BH18-14: When Wood personnel returned to site on 28 August 2018 to record water-level readings, it was discovered that the 50 mm diameter slotted PVC standpipe at BH18-14 had either sloughed in or broken at a depth of 1.3 m below existing grade.

It should be recognized that the level of the groundwater table is dependent on meteorological cycles and surface drainage on a regional scale. Higher groundwater levels than those observed in this investigation may be encountered following spring thaw and periods of prolonged precipitation. Seasonal fluctuations under normal conditions are expected to be  $\pm 1.0$  m from the observed groundwater level although greater fluctuations are also possible.



### 3.3 Water Soluble Sulphates

Three (3) water soluble sulphate concentration tests were performed on soil samples obtained from the site. **Table 2** below summarizes the results of the water soluble sulphate tests, indicating percent water soluble sulphates by dry weight of soil.

**Table 2: Water Soluble Sulphate Concentrations**

Borehole	Depth (m)	Material Type	Water-Soluble Sulphate (%)
BH18-02	0.8	Sand	< 0.01
BH18-03	9.8	Sand	0.06
BH18-05	2.0	Sand	< 0.01

These values are considered low and indicate a low potential for sulphate attack on concrete that comes in contact with native soils at this site.

### 3.4 Permafrost

Yellowknife lies within the discontinuous zone of permafrost. That is, permafrost at undisturbed sites is usually found except beneath lakes and rivers. Within the test holes drilled for this investigation, permafrost was not encountered. As this development is within the City of Yellowknife (i.e. disturbed site) permafrost is unlikely.

If permafrost is encountered Wood should be contacted to review the site conditions.

## 4.0 Geotechnical Appraisal

Relative to the proposed development, the subsurface soil and groundwater conditions observed in the test borings are considered only fair.

For the proposed school, the structural components may be supported on shallow foundations bearing on the very dense sand. The depth to bedrock, and variability of the depth to bedrock encountered would make founding the footings on the bedrock unfeasible. Adequate soil cover, or insulation, will be required to maintain perimeter footings below the seasonal frost penetration depth. As the building, will be permanently heated, the interior footings may be founded at higher levels in the sand.

Straight-shaft drilled cast-in-place concrete piles are not considered to be feasible due to the anticipated difficulty of installation with the high water table and sandy conditions encountered. Driven steel piles are unlikely to be cost competitive.

The geotechnical design parameters are presented in this report are limed to strip and square footings.

The existing sand subgrade soil is suitable to provide support for concrete floor slabs and pavements. The high water table encountered would cause difficulty if grades are to be lowered across the site, making crawl spaces unfeasible.

## 5.0 Recommendations

### 5.1 Site Preparation, Grading and Drainage

#### 5.1.1 Subgrade Preparation

The areas for the proposed structures, parking and roadways should be stripped of all fill and organic soil. Fill required to achieve the required top-of-subgrade elevation should consist of an engineered fill as described in Subsection 5.1.2. Where loose, soft or disturbed areas are identified, the area should be excavated to expose a stable subgrade and then should be backfilled with engineered fill.

The sand should be proof-rolled to check for soft spots. The proof-roll should be conducted with an axle load of 80 kN to check for soft, loose or non-uniform areas. Any such areas detected should be over-excavated to a maximum depth of 300 mm and replaced with engineered fill material. Alternatively, if high water tables do not allow for area to be over excavated geotextile and/or geogrid may be required.

#### 5.1.2 Engineered Fill

Engineered fill may be required to bring the building floor slabs, sidewalks and pavement subgrade areas up to design grade. Engineered fill should preferably consist of well-graded gravel, or alternatively an imported, low to medium plastic clay.

Where clay is used as engineered fill, it should be at moisture contents within  $\pm 2$  percent of the optimum moisture content (OMC) at the time of compaction. Clay should be placed in compacted lift thicknesses not exceeding 150 mm, with each lift compacted to a minimum of 98 percent of the standard Proctor maximum dry density (SPMDD). The local sand is suitable for use as engineered fill, provided that it is suitably moisture conditioned. The sand and silty sand at the site will be sensitive to small changes in moisture content, and ideally the moisture content during compaction should be maintained within about one percent of optimum moisture content.

If gravel is to be used for engineered fill, as a minimum it should consist of 80 mm minus pit run. It should be uniformly compacted to minimum of 98 percent SPMDD for grade supported slab areas and 95 percent SPMDD for pavement areas, and placed in compacted lift thicknesses not exceeding 150 mm. Gradation limits for the pit run, for use as engineered fill are provided in **Table 3**.

**Table 3: Gradation Limits for 80 mm Minus Pit Run**

Sieve	Percent Passing
80 mm	100
50 mm	55-100
25 mm	38-100
16 mm	32-85
4.75 mm	20-65
0.315 mm	6-30
0.08 mm	2-10

All fill soils should be free from any organic materials, contamination, deleterious construction debris, and stones greater than 80 mm in diameter. Environmental screening should be conducted on any fill source of unknown origin and history. Fill construction and compaction should be monitored on a full-time basis, including regular field density testing during placement at a frequency of a minimum of 1 test per 300 m<sup>2</sup> per lift.

The engineered fill should extend at least 1 m beyond the footprint of any building footprint or pavement. Fill soils should be compacted uniformly over the area that will provide support for building structural elements or pavement in order to reduce potential for differential settlement. Fill should not be frozen at the time of placement; nor should the fill be placed on a frozen subgrade or be allowed to freeze during construction.

### **5.1.3 Drainage**

The prepared subgrade should be shaped to reduce the potential for ponding of water under the structure footprint. Excess water should be drained or pumped from the site as quickly as possible, both during construction and over the long-term use of the site.

Design finished grades within 2 m of the building perimeter should provide surface drainage at approximately a 2.0 percent grade away from the structure. The upper 0.3 m of backfill around the buildings should consist of compacted clay, asphalt or concrete to act as a seal against the ingress of runoff water. The clay should extend for a distance of 3 m around the building and should be graded at a slope of 2 percent away from the building. Roof and other drains should discharge at least 2 m clear of the building perimeter.

### **5.1.4 Winter Construction**

Fill placement and compaction during the winter months is not recommended since the required degrees of compaction cannot be attained using frozen fill soils or fill which appears to be unfrozen but is at subfreezing temperatures. Even gravels, which give an appearance of being not affected by frozen conditions, can contain ice crystals which limit the degree of compaction that could be attained. A high degree of compaction during the winter months can only be achieved in fill soils that are unfrozen and are not allowed to freeze during placement and compaction. This would necessitate that all fill soils are unfrozen.

It should also be noted that unless the fill placement area is hoarded and heated, the addition of water to the fill to promote its compaction would not be possible at freezing temperatures.

## **5.2 Shallow Foundations**

### **5.2.1 Design**

The native dense to very dense sand found at the site is a suitable bearing medium for support of strip and square footings for the proposed building.

Footings founded in the dense to very dense sand may be designed using recommended serviceability limit state (SLS) bearing pressure values of 150 kPa and 175 kPa for strip and square footings respectively. The recommended serviceability bearing resistance values are based on limiting the settlement to less than 25 mm, and are applicable to strip footings to a maximum dimension of 1.2 m wide or square footings measuring 2 x 2 m. If very strict settlement tolerances are required, or if larger footings are proposed, the footing sizes and settlement potential should be reviewed by Wood.

Perimeter footings supporting heated structures should have a minimum of 1.4 m of soil cover, or equivalent insulation, for frost protection. Interior footings supporting a heated structure do not require soil cover for frost protection but should be based a minimum of 0.3 m below the base of the floor slab to provide confinement of the subgrade soil. Footings for unheated facilities should be provided a minimum of 3.0 m of soil cover, or equivalent insulation, for frost protection.

The corresponding unfactored ultimate limit state (ULS) bearing pressure values are 450 kPa and 525 kPa for strip and square footings, respectively. The unfactored ULS bearing pressure should be multiplied by a

geotechnical resistance factor of 0.5 to arrive at the factored ULS bearing values, per the recommendations in the current Canadian Foundation Engineering Manual.

### 5.2.2 Footing Construction

The following geotechnical recommendations are provided for the construction of shallow footings:

- Footing excavations should be based on undisturbed, native, dense to very dense sand.
- The bearing surface of each footing should be excavated in a manner to minimize disturbance of the subgrade. Any loose soils at the base of the excavation should be removed from below the footing bases.
- A lean concrete mud slab should be cast over top of the exposed sand at the base of each footing excavation to mitigate potential disturbance effects from construction foot traffic during placement of the reinforcing steel.
- The bearing surfaces should be protected from rain, snow and the ingress of free water, as the foundation soils may experience loss of bearing strength should they be subjected to increases in moisture. In this case, softened soils would have to be removed and the footings extended to suitable bearing soils.
- The foundation soils beneath the footings must not be allowed to freeze during construction or during the service life of the building. Footings founded on frozen soil during construction may settle when the founding soils thaw. Bearing soils that become frozen during construction should be removed and replaced with concrete fill, or the embedment depths should be extended to unfrozen native soils.
- It is possible that during construction, groundwater seepage or rainfall may be encountered. In either of these cases, drainage of footing excavations will be required to facilitate footing construction. It is anticipated that dewatering can be achieved by gravity drainage into small sumps or perimeter ditches within the excavations, which could be pumped out as required. The crests of the foundation excavations should be graded such as to direct surface water runoff away from the excavations.
- A geotechnical engineer or qualified technician should observe the exposed bearing surface prior to casting the mud slab to check that the exposed subgrade is competent soil as identified in the geotechnical report, and is suitably prepared, as discussed above.

### 5.3 Excavations

For this project, it is envisaged that excavations will be required for service trenches. The following recommendations are provided, assuming that the excavation depth will not exceed 4 m below existing grade. Based on this assumed excavation depth and the soil conditions encountered at the borehole locations, such excavations may encounter sand and bedrock. For open short-term excavations, less than 1 m in depth, near-vertical excavation side slopes may be considered. For open unsupported short-term excavations, deeper than 1m, the side slopes should be cut back at inclinations no steeper than 1H:2V. Flatter inclinations may be required in localized zones. Short term excavations are those which will remain open for a period of 2 months or less.

The stability of the excavation is highly dependent on the efficiency of the surface water and groundwater control measures adopted at the time of construction. Groundwater was encountered in the sand subgrade at shallow depths in the boreholes at the site. Although the volume groundwater and rate of flow into excavations at the site are not known, the volume could potentially be high in a sand soil. If the rate of inflow is greater than can be handled with temporary sumps and submersible pumps, then other measures such as well points could be required. Surface grading should be undertaken so that surface water is not allowed to pond adjacent to the excavation and to prevent run-off water from entering the excavation. With a sloped excavation sidewall, some sloughing may be expected and periodic cleaning of debris at the base of excavation may be required.



As a minimum, excavations should comply with regulations set forth by the applicable local regulations. The stability of all excavations should be monitored by the excavation contractor on an on-going basis. Where tension cracks, or ravelling soils are detected, these conditions should be brought to the immediate attention of Wood so that engineered solutions to the problem areas can be appropriately determined.

Stockpiles of materials and excavated soil should be placed away from the slope crest by a distance equal to the depth of excavation. Similarly, wheel loads should be kept back at least 1 m from the crest of the excavation. Surface drainage should be directed away from crest of the excavation.

The stability of excavation slopes generally decreases with time, and therefore construction should be directed at minimizing the length of time the excavation is left open.

## 5.4 Backfill

In areas where subgrade support is required (for example below floor slabs, pavements, etc.) the backfill should consist of engineered fill in accordance as with the recommendations given in Section 5.1.2. For fill compacted to 98 percent of the SPMDD, the settlement due to re-orientation of soil particles (i.e. self-weight) would be in the range of 0.5 to 1 percent of the height of fill. Where settlement of surface facilities can be tolerated, the degree of compaction may be reduced backfill. For backfill placed at degrees of compaction between 90 and 95 percent of the SPMDD, settlements in the range of 5 percent to 1.5 percent, respectively, of the fill height may occur.

## 5.5 Floor Slabs

### 5.5.1 Subgrade Preparation

Slab-on-grade floors may be supported on new engineered fill underlain by the sand. Preparation of the exposed subgrade should be undertaken as described in Subsection 5.1.1.

The slab-on-grade should be allowed to move independently of footings, columns and exterior slabs. A minimum thickness of 200 mm of clean, well-graded crushed gravel is recommended beneath the grade supported floor slab. Coarse material greater than 50 mm in diameter should be avoided directly beneath the floor slab to prevent stress concentrations in the slab. The gravel base course should be compacted to a uniform dry density of 100 percent of SPMDD within  $\pm 2\%$  of the OMC. A recommended typical gradation for stable granular material, for use as base course under floor slabs is provided in **Table 4**.

**Table 4: Gradation Requirement for Granular Backfill**

Sieve	Percent Passing
20 mm	100
10 mm	35-77
5 mm	15-55
1.25 mm	0-30
0.08 mm	0-10

The percent fracture by weight (2 faces) should be at least 40 percent. Other appropriate materials, which fall outside the above recommended gradation limits, may be suitable and should be evaluated by a geotechnical engineer prior to use.

Grade supported floor slabs should be allowed to "float" on a prepared subgrade and be independent of structural components supported by building foundations. Equipment and placed on floor slabs should be designed to allow re-leveling if the equipment is sensitive to settlement. Interior walls supported on floor slabs should be designed to accommodate up to 25 mm of settlement or heave.

### 5.5.2 Drainage Measures

As groundwater was encountered within the boreholes drilled at the site, perimeter weeping tile is recommended along the exterior footing perimeter. The weeping tile is recommended as a measure to intercept and dispose of surface runoff that may infiltrate along the soil/concrete interface. The weeping tile system should consist of a minimum 150 mm diameter perforated PVC pipe. The pipe should be placed in a trench backfilled by free draining 40 mm minus washed gravel. The trench should be at least 300 mm wide and 300 mm deep and lined with a non-woven geotextile filter such as Nilex C24, to control migration of fines into the lines. The weeping tile should drain to a sump with a pumped discharge to the storm sewer. The drainage gravel should correspond to the gradation outlined in **Table 5**.

In specific areas an under-slab drainage system may be required to protect portions of building from potential groundwater infiltration. An under-slab drainage system consists of perforated drains installed below the floor slab, which are positively drained to a central pumped sump or sumps. The requirement for an under-drain system should be assessed during excavation of building foundations if groundwater is encountered. The following paragraphs outline the general requirements for an under-slab drainage system, in the event that one is required in some areas.

The under-slab drainage system where required below the floor slab should consist of minimum 150 mm diameter perforated PVC pipes. The pipes should be placed in trenches backfilled with free draining 25 mm minus washed gravel. The trenches should be at least 300 mm wide and 300 mm deep and lined with a non-woven geotextile filter to control migration of fines into the lines. A Nilex C24 geotextile, or equivalent, is recommended. Above the trenches and beneath the slab there should be a 200 mm thick layer of drainage gravel. The drainage gravel should correspond to the following gradation:

**Table 5: Recommended Gradation for the Drainage Gravel**

Sieve	Percent Passing
25 mm	100
20 mm	60-80
15 mm	30-60
10 mm	10-30
5 mm	0-10
2.36 mm	0-5

The drainage gravel should be compacted to at least 98 percent of SPMDD.

The design capacity of the under-drainage system should be assessed during excavation for footings and service trenches, when groundwater conditions can be observed directly.

Both the weeping tile system and any under-slab drainage system should be provided with cleanouts in order to flush the lines in the event of line siltation. The actual design of the subdrainage system should be developed by the mechanical designer/contractor using the above recommendations as a guideline.

### 5.5.3 Exterior Grade Supported Sidewalks and Concrete Aprons

Subgrade preparation for in sidewalk and concrete aprons should be carried out as recommended in Subsection 5.1.1. The sand subgrade is considered to be moderately frost susceptible particularly with the shallow groundwater levels encountered at the site, and may develop ice lenses and undergo volume change (heave). Therefore, it will be important to provide adequate site drainage as per Subsection 5.1.3. Exterior sidewalks and apron slabs should be free-floating and should not be dowelled into grade beams, or interior slabs.



Consideration can be given to installing rigid insulation below the sidewalks or aprons (driveways) if frost heave is a concern. Additional measures to reduce the risk of frost heave include sloping the aprons or sidewalks away from the building and sealing the interface between the basement walls and the exterior concrete flatwork to limit seepage of surface runoff into the subgrade soils. Where pavement areas are adjacent to walls or grade beams, a separation strip should be installed at the interface.

## 5.6 Pavements

The pavement structures and construction procedure recommendations provided in this section for light traffic are applicable for access roadways and parking areas typically used by passenger cars and light trucks with occasional use by single axle delivery trucks, waste disposal trucks, etc. In areas where heavier cargo truck traffic is expected, such as drive lanes, the heavier traffic pavement specifications should be used.

Prior to placing base gravel, the subgrade should be prepared as outlined in Subsection 5.1.1, and proof rolled with a loaded tandem axle truck. If soft subgrades are encountered during the proof roll, subgrade improvement should be carried out, such as over-excavation and backfill with a thicker gravel fill and/or geotextiles or geogrids, the extent of which would be best determined during construction. **Table 6** outlines the recommended light vehicle and heavy vehicle pavement sections for access roadways, parking lots and aprons.

**Table 6: Pavement Sections**

Pavement Component	Minimum Thicknesses (mm)	
	Light Traffic/ Parking Area (assumed $1.44 \times 10^4$ ESAL's <sup>1</sup> )	Heavy Truck Traffic/Drive Lanes (assumed $3.6 \times 10^4$ ESAL's)
Hot Mix Asphalt	75	100
Base Course Crushed Granular <sup>2</sup> (20 mm minus)	250	300

Note(s)

Alberta Transportation Specifications:

1. Equivalent Single Axle Loads over 20-year design period
2. AT Designation 2 Class 20 (see Table 4)

Outlined below are additional construction recommendations pertaining to the pavement sections:

- The granular base course should be placed in maximum 150 mm thick lifts (or reduced lift thicknesses as governed by the compaction equipment) and uniformly compacted to a minimum 100 percent of SPMDD at  $\pm 2$  percent of OMC.
- All asphalt should conform to, and be placed in accordance with, the current applicable local asphalt specifications.

Concrete pavement sections should be provided for any areas where the front wheels of garbage trucks will bear during unloading of dumpsters, and for any areas where trailer "dollies" will bear on the pavement. Asphalt pavement used in such areas is at high risk of rutting, and normally develops ruts and cracks within a short time.

## 5.7 Concrete Type

As per CSA A23.1-09, measured concentrations of water soluble sulphates in soil samples from the current investigation were less than the minimum required for sulphate resistant cement. Therefore, General Use (GU) type cement may be used for construction. It should be recognized that structural requirements and other considerations may necessitate additional criteria be used for determining the type of subsurface



concrete to be used. To enhance durability, an appropriate quantity of entrained air as per CSA specification CAN/CSA-A23.1-04, Clause 4.1.1.3, is recommended for all concrete exposed to freezing and thawing at this site.

## **6.0 Geotechnical Testing and Inspection**

All engineering design recommendations presented in this report are based on the limited number of boreholes advanced at the site, and on the assumption that an adequate level of inspection will be provided during construction and that all construction will be carried out by a suitably qualified contractor experienced in foundation and earthworks construction. An adequate level of inspection is considered to be:

- for footing foundations: review of foundation drawings and inspection of all foundation subgrades by the geotechnical engineer prior to placement of foundation concrete; and
- for earthworks: full time monitoring and compaction testing.

Wood requests the opportunity to review the design drawings and monitor the installation of the new foundation to confirm that the recommendations have been correctly interpreted. Wood would be pleased to provide any further information that may be needed during design and to advise on the geotechnical aspects of specifications for inclusion in contract documents.

## 7.0 Closure

Recommendations presented herein are based on a geotechnical evaluation of the findings at the 14 borehole locations drilled during the field investigation at the site. If conditions other than those reported are noted during subsequent phases of the work, Wood should be notified and given the opportunity to review the current recommendations considering any new findings. Recommendations presented herein may not be valid if an adequate level of inspection is not provided during construction, or if relevant building code requirements are not met.

Soil conditions, by their nature, can be highly variable across a construction site. The placement of fill and prior construction activities on a site can contribute to variable near surface soil conditions. A contingency amount should be included in the construction budget to allow for the possibility of variations in soil conditions, which may result in modifications of the design, and/or changes in construction procedures.

This report has been prepared for the exclusive use of the Government of the Northwest Territories for specific application to the development described within this report. Any use that a third party makes of this report, or any reliance or decisions based on this report are the sole responsibility of those parties. It has been prepared in accordance with generally accepted soil and foundation engineering practices. No other warrantee is given.

Respectfully submitted,

**Wood Environment & Infrastructure Solutions,  
a division of Wood Canada Limited**

Tyson Tremblay, P.Eng. (AB)  
Senior Geotechnical Engineer

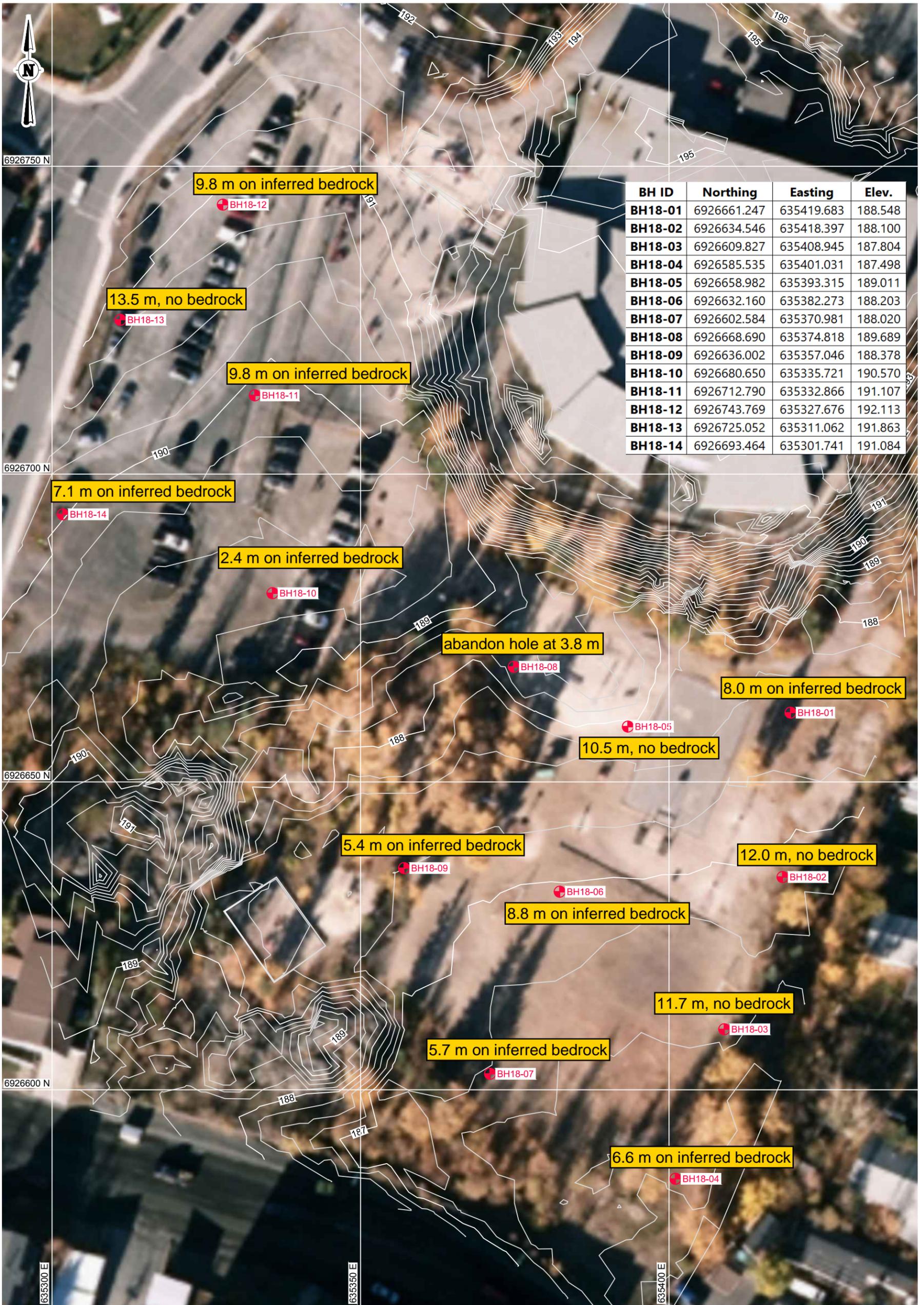


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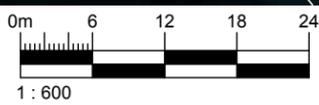
<b>PERMIT TO PRACTICE</b>	
Wood Environment & Infrastructure Solutions, a Division of Wood Canada Limited	
Signature	
Date	<u>Sept. 24/18</u>
<b>PERMIT NUMBER: P 047</b>	
NT/NU Association of Professional Engineers and Geoscientists	

# **Appendix A**



BH ID	Northing	Easting	Elev.
BH18-01	6926661.247	635419.683	188.548
BH18-02	6926634.546	635418.397	188.100
BH18-03	6926609.827	635408.945	187.804
BH18-04	6926585.535	635401.031	187.498
BH18-05	6926658.982	635393.315	189.011
BH18-06	6926632.160	635382.273	188.203
BH18-07	6926602.584	635370.981	188.020
BH18-08	6926668.690	635374.818	189.689
BH18-09	6926636.002	635357.046	188.378
BH18-10	6926680.650	635335.721	190.570
BH18-11	6926712.790	635332.866	191.107
BH18-12	6926743.769	635327.676	192.113
BH18-13	6926725.052	635311.062	191.863
BH18-14	6926693.464	635301.741	191.084

LEGEND:  
 BOREHOLE LOCATION



**DRAFT**

CLIENT:  
**GOVERNMENT OF THE NORTHWEST TERRITORIES**

**wood.** Environment & Infrastructure Solutions  
 4920-51 STREET, YELLOWKNIFE, NT, X1A 3S8  
 PHONE 867-920-4140

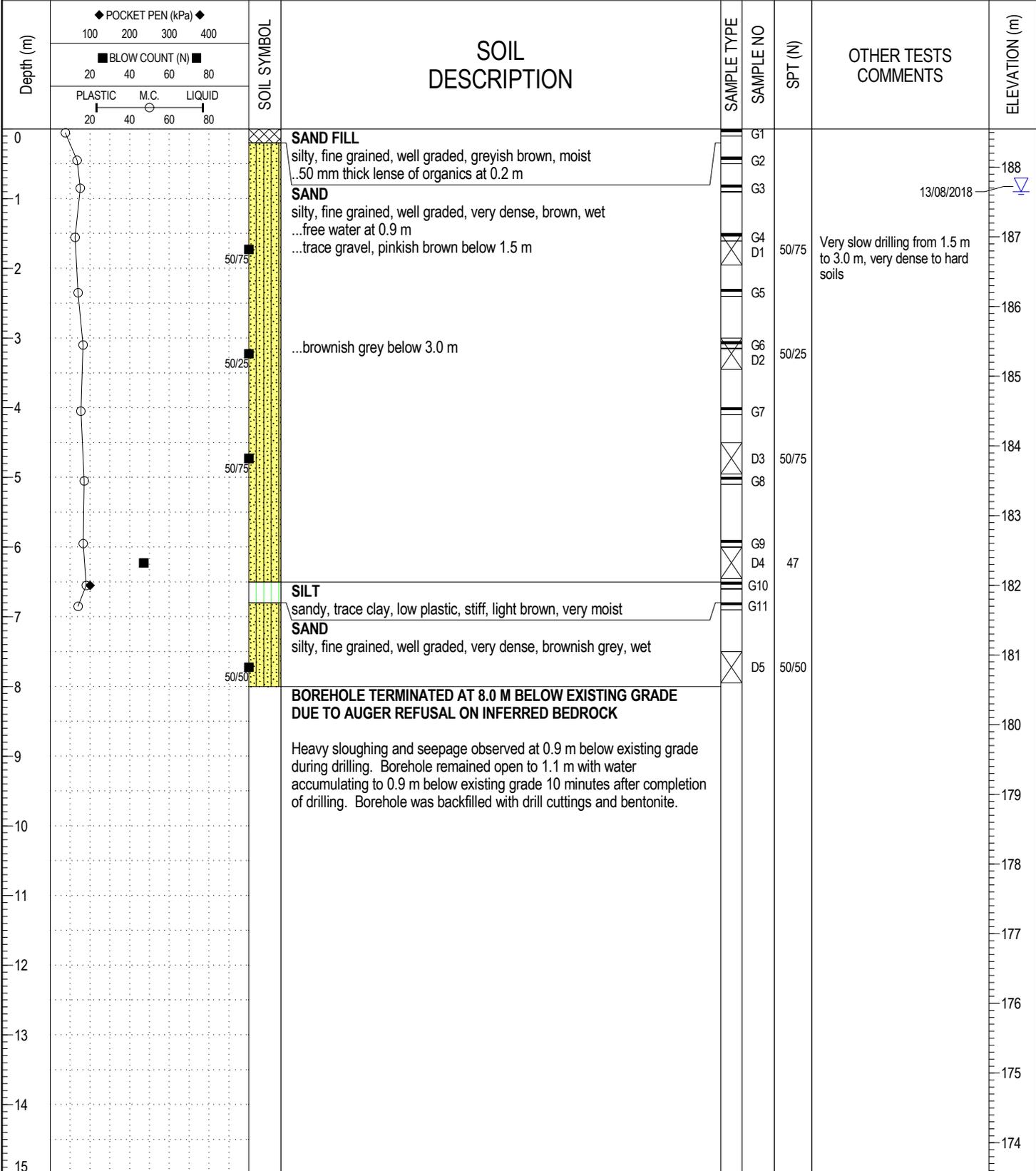
DWN BY: BGR  
 CHK'D BY: KN  
 DATUM: NAD 83 CSRS (2010.0)  
 PROJECTION: UTM Zone 11  
 SCALE: 1:600

PROJECT: **ECOLE J.H. SISSONS SCHOOL GEOTECHNICAL INVESTIGATION AND ENVIRONMENTAL SITE ASSESSMENTS**

TITLE: **BOREHOLE LOCATION PLAN**

DATE: SEPTEMBER, 2018  
 PROJECT No.: YX18006  
 REV. No.: A  
 FIGURE No.: **FIGURE 1**

Government of the Northwest Territories	Ecole JH Sissons School Geotechnical Investigation & ESA	BOREHOLE NO.: <b>BH18-01</b>
Landmark Drilling Services Ltd	SITE: 5700 51a Ave, Yellowknife, NT	PROJECT NO.: YX18006
Solid Stem Augers	NAD83(CSRS) / UTM zone 11N N:6926661 E:635420	ELEVATION: 188.5 m
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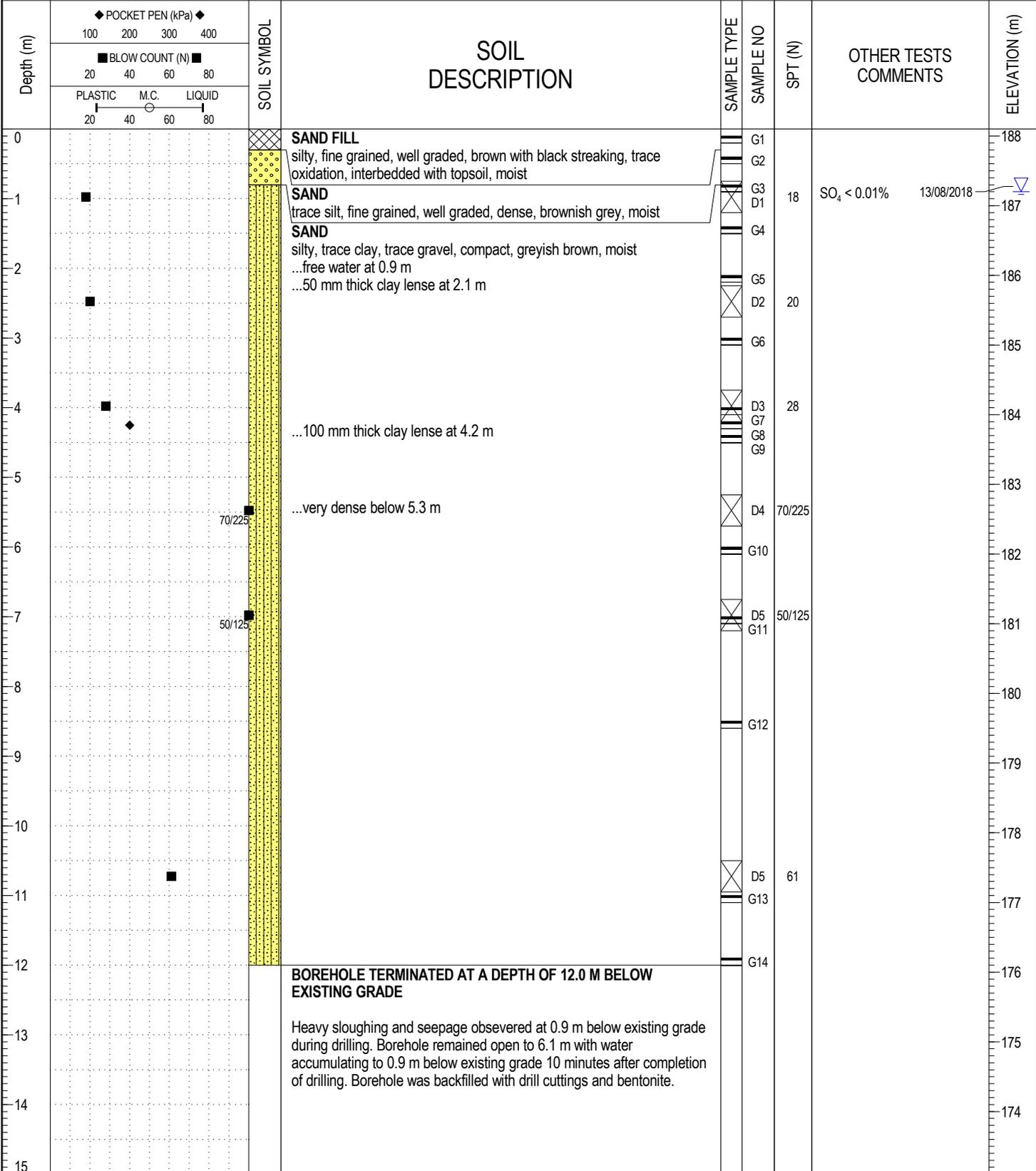


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5681 - 70 Street NW  
Edmonton, Alberta, T6B 3P6

ENTERED BY: CT  
LOGGED BY: CT  
REVIEWED BY: TT

COMPLETION DEPTH: 8 m  
COMPLETION DATE: 13/8/18

Government of the Northwest Territories	Ecole JH Sissons School Geotechnical Investigation & ESA	BOREHOLE NO.: <b>BH18-02</b>
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COMPLETION DEPTH: 12 m  
COMPLETION DATE: 13/8/18

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Landmark Drilling Services Ltd		SITE: 5700 51a Ave, Yellowknife, NT		PROJECT NO.: YX18006			
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BACKFILL TYPE		Bentonite	Pea Gravel	Slough	Grout	Drill Cuttings	Sand

Depth (m)	SOIL SYMBOL	SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NO	SPT (N)	OTHER TESTS COMMENTS	ELEVATION (m)
0	[Symbol: Sand]	<b>SAND</b> trace silt, trace clay, fine grained, well graded, dense, light brown, very moist	[Symbol: SPT]	G1	17		187
1		...wet below 1.0 m		D1			
2		...free water at 1.6 m		G3			
3				D2			
4				G5			
5	[Symbol: Sand]	<b>SAND</b> silty, trace gravel, fine grained, well graded, very dense, greyish brown, very moist	[Symbol: SPT]	G6	28	Switch to hollow stem augers at 3.8 m	185
6		...no gravel below 3.5 m		D3			
7				G4			
8				D4			
9				G2			
10				D5			
11				G1			
12				D6			
13	[Symbol: Borehole Termination]	<b>BOREHOLE TERMINATED AT A DEPTH OF 11.7 M BELOW EXISTING GRADE</b>	[Symbol: SPT]	D7	59	SO <sub>4</sub> = 0.06%	178
14		Heavy sloughing at 1.4 m and seepage at 1.6 m below existing grade was observed during drilling. Borehole remained open to 1.4 m with water accumulating to 0.2 m below existing grade 10 minutes after completion of drilling. Borehole was backfilled with drill cuttings and bentonite.		D8			
15				G3			
				D1			
				G1			

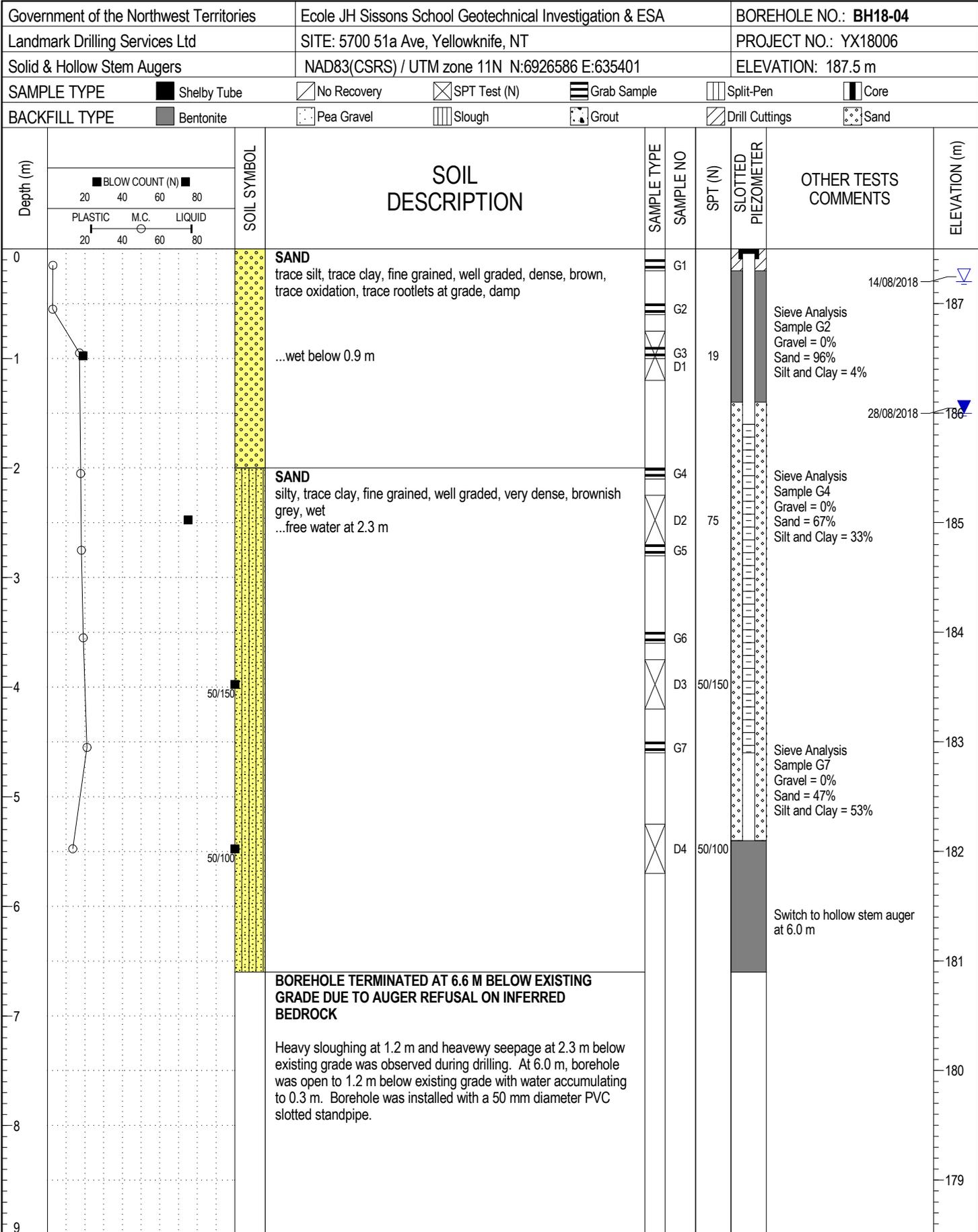
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COMPLETION DEPTH: 11.7 m  
COMPLETION DATE: 14/8/18  
Page 1 of 1



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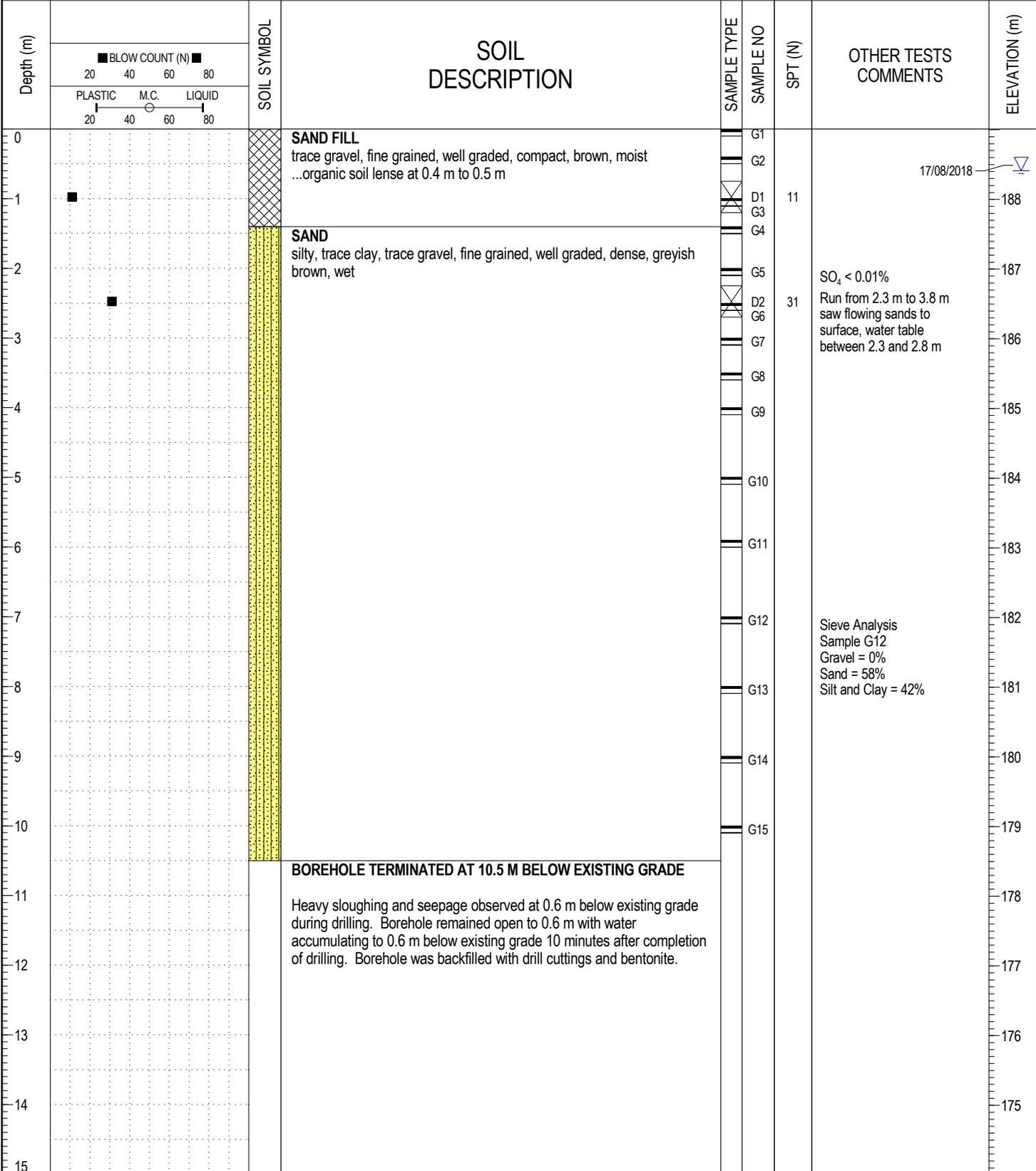


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 COMPLETION DATE: 14/8/18

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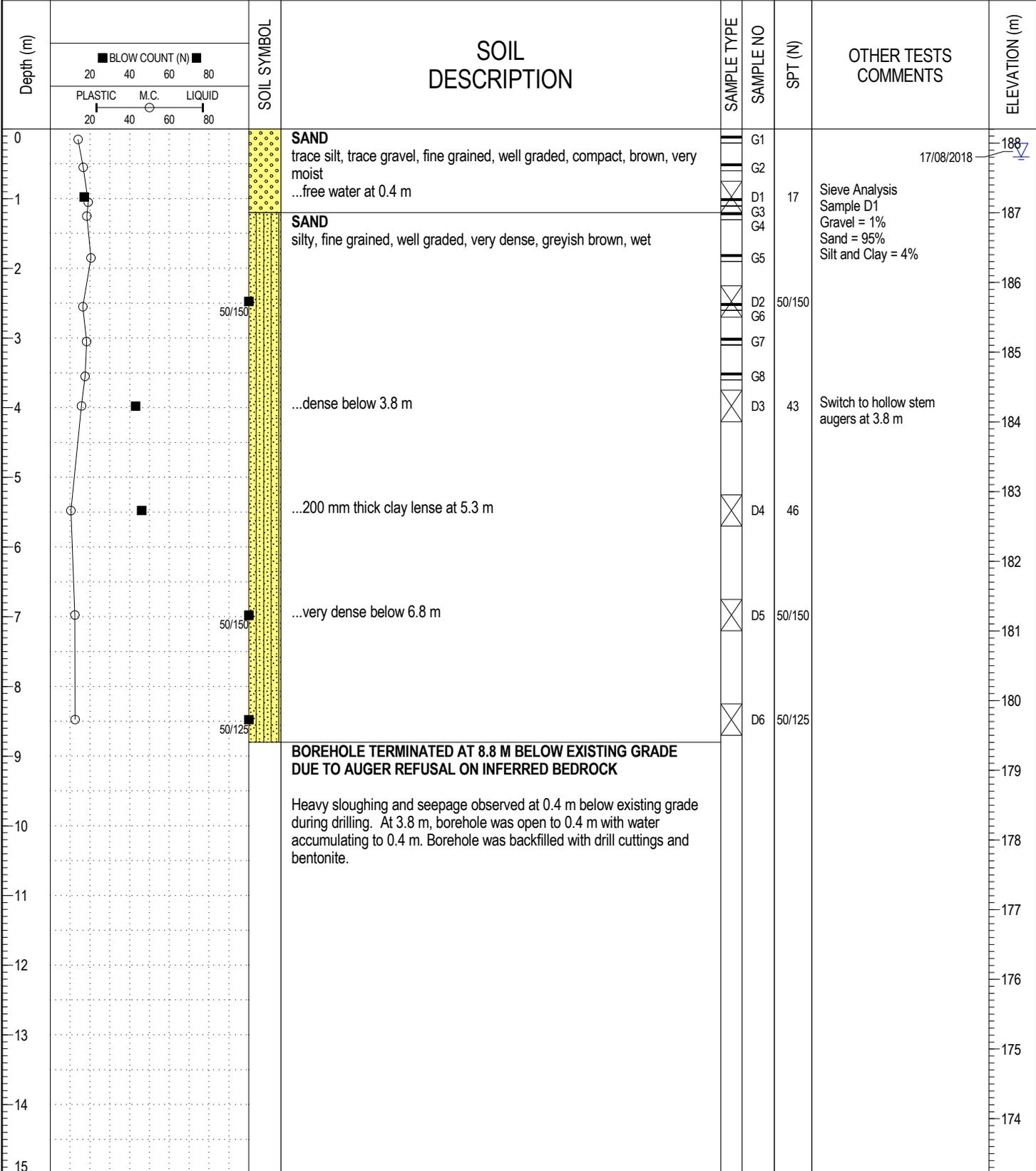


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COMPLETION DEPTH: 10.5 m  
COMPLETION DATE: 17/8/18

Government of the Northwest Territories	Ecole JH Sissons School Geotechnical Investigation & ESA	BOREHOLE NO.: <b>BH18-06</b>
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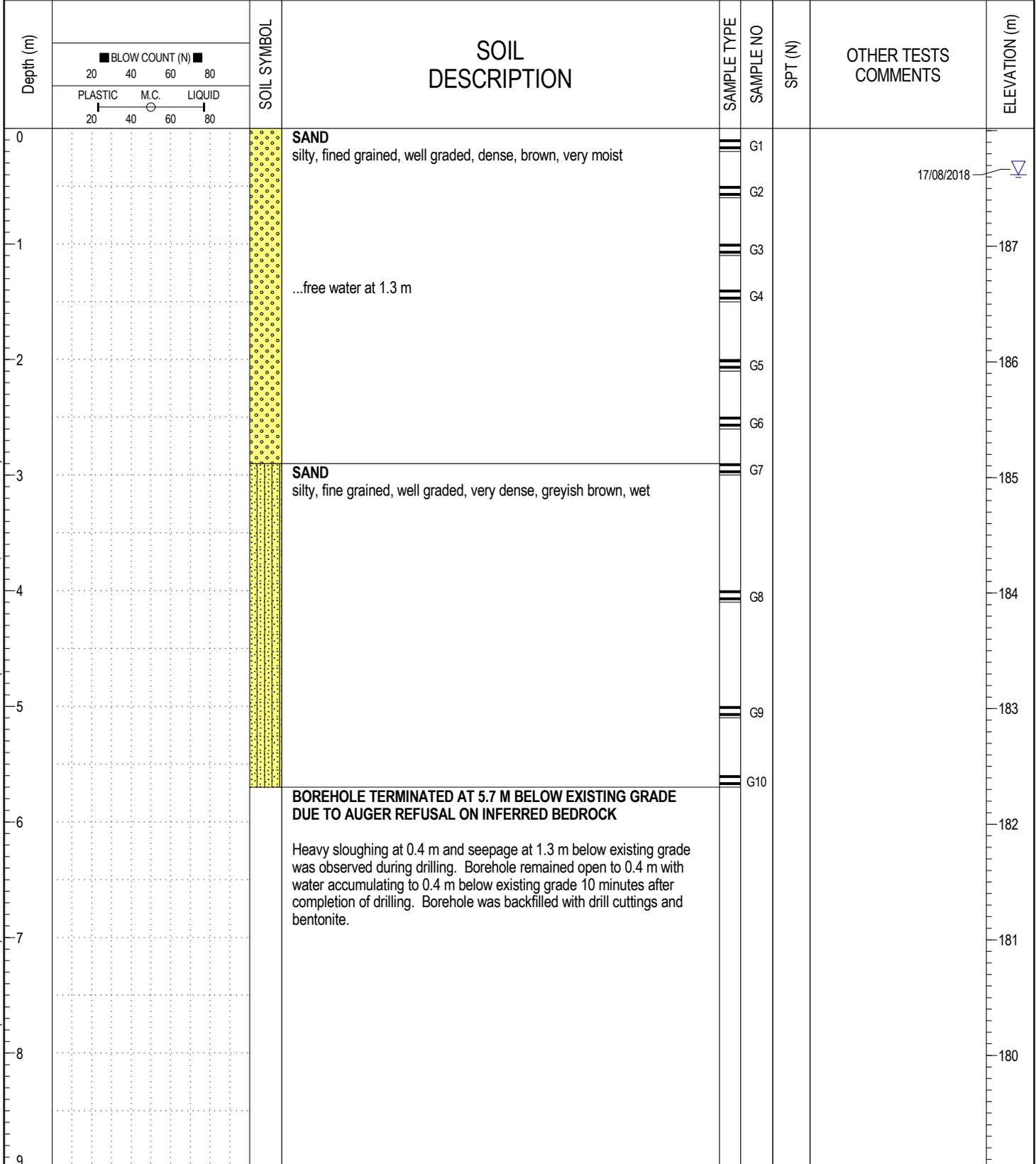


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5681 - 70 Street NW  
Edmonton, Alberta, T6B 3P6

ENTERED BY: CT  
LOGGED BY: CT  
REVIEWED BY: TT

COMPLETION DEPTH: 8.8 m  
COMPLETION DATE: 17/8/18

Government of the Northwest Territories	Ecole JH Sissons School Geotechnical Investigation & ESA	BOREHOLE NO.: <b>BH18-07</b>
Landmark Drilling Services Ltd	SITE: 5700 51a Ave, Yellowknife, NT	PROJECT NO.: YX18006
Solid Stem Augers	NAD83(CSRS) / UTM zone 11N N:6926603 E:635371	ELEVATION: 188 m
SAMPLE TYPE	<input checked="" type="checkbox"/> Shelby Tube <input type="checkbox"/> No Recovery <input type="checkbox"/> SPT Test (N) <input type="checkbox"/> Grab Sample <input type="checkbox"/> Split-Pen <input type="checkbox"/> Core	
BACKFILL TYPE	<input checked="" type="checkbox"/> Bentonite <input type="checkbox"/> Pea Gravel <input type="checkbox"/> Slough <input type="checkbox"/> Grout <input type="checkbox"/> Drill Cuttings <input type="checkbox"/> Sand	



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COMPLETION DEPTH: 5.7 m  
COMPLETION DATE: 17/8/18  
Page 1 of 1

Government of the Northwest Territories		Ecole JH Sissons School Geotechnical Investigation & ESA			BOREHOLE NO.: <b>BH18-08</b>				
Landmark Drilling Services Ltd		SITE: 5700 51a Ave, Yellowknife, NT			PROJECT NO.: YX18006				
Solid & Hollow Stem Augers		NAD83(CSRS) / UTM zone 11N N:6926669 E:635375			ELEVATION: 189.7 m				
SAMPLE TYPE		<input checked="" type="checkbox"/> Shelby Tube	<input type="checkbox"/> No Recovery	<input checked="" type="checkbox"/> SPT Test (N)	<input type="checkbox"/> Grab Sample	<input type="checkbox"/> Split-Pen	<input type="checkbox"/> Core		
BACKFILL TYPE		<input checked="" type="checkbox"/> Bentonite	<input type="checkbox"/> Pea Gravel	<input type="checkbox"/> Slough	<input type="checkbox"/> Grout	<input checked="" type="checkbox"/> Drill Cuttings	<input type="checkbox"/> Sand		
Depth (m)			SOIL SYMBOL	SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NO	SPT (N)	OTHER TESTS COMMENTS	ELEVATION (m)
0				<b>SAND FILL</b> trace silt, trace gravel, fine grained, well graded, light brown, damp <b>SAND</b> trace silt, fine grained, well graded, compact, brown to reddish brown, moist	G1				189
1				...some gravel, coarse grained, well graded, blackish brown (no odor, no staining) below 1.2 m	D1 G3		12		
2				...free water at 2.2 m	G5				188
3				<b>SAND</b> silty, fine grained, well graded, very dense, greyish brown, wet	D2		50/50		187
4				<b>BOREHOLE TERMINATED AT 3.8 M BELOW EXISTING GRADE</b>  Heavy sloughing and seepage observed at 2.2 m below existing grade during drilling. At 3.8 m, borehole was open to 2.2 m with water accumulating to 2.2 m. Borehole was backfilled with drill cuttings and bentonite.	G6			Switch to hollow stem augers at 3.8 m	186
5									185
6									184
7									183
8									182
9									181

15/08/2018

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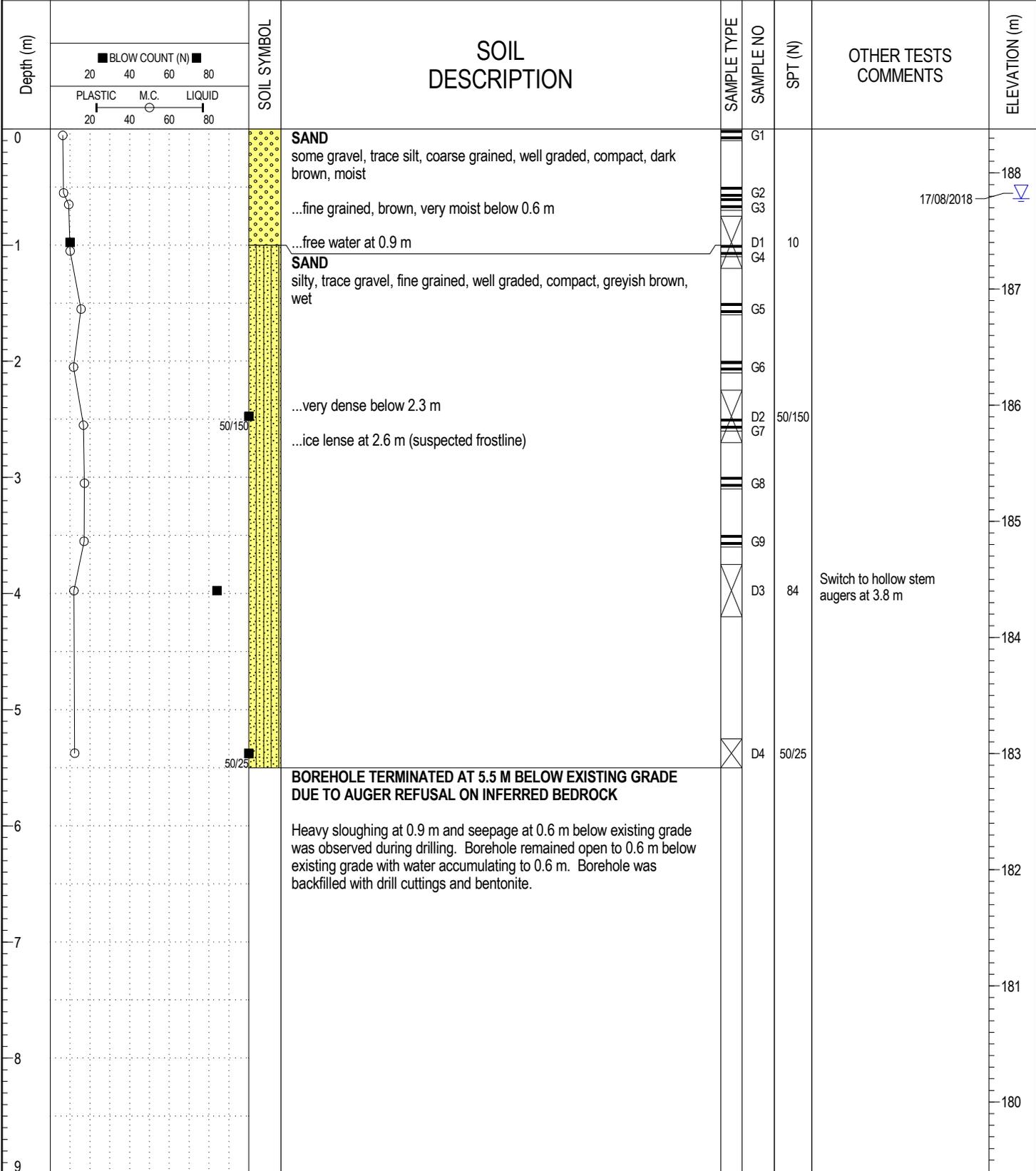


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COMPLETION DEPTH: 3.8 m  
 COMPLETION DATE: 15/8/18  
 Page 1 of 1

Government of the Northwest Territories	Ecole JH Sissons School Geotechnical Investigation & ESA	BOREHOLE NO.: <b>BH18-09</b>
Landmark Drilling Services Ltd	SITE: 5700 51a Ave, Yellowknife, NT	PROJECT NO.: YX18006
Solid & Hollow Stem Augers	NAD83(CSRS) / UTM zone 11N N:6926636 E:635357	ELEVATION: 188.4 m
SAMPLE TYPE	<input checked="" type="checkbox"/> Shelby Tube <input type="checkbox"/> No Recovery <input type="checkbox"/> SPT Test (N) <input type="checkbox"/> Grab Sample <input type="checkbox"/> Split-Pen <input type="checkbox"/> Core	
BACKFILL TYPE	<input checked="" type="checkbox"/> Bentonite <input type="checkbox"/> Pea Gravel <input type="checkbox"/> Slough <input type="checkbox"/> Grout <input type="checkbox"/> Drill Cuttings <input type="checkbox"/> Sand	



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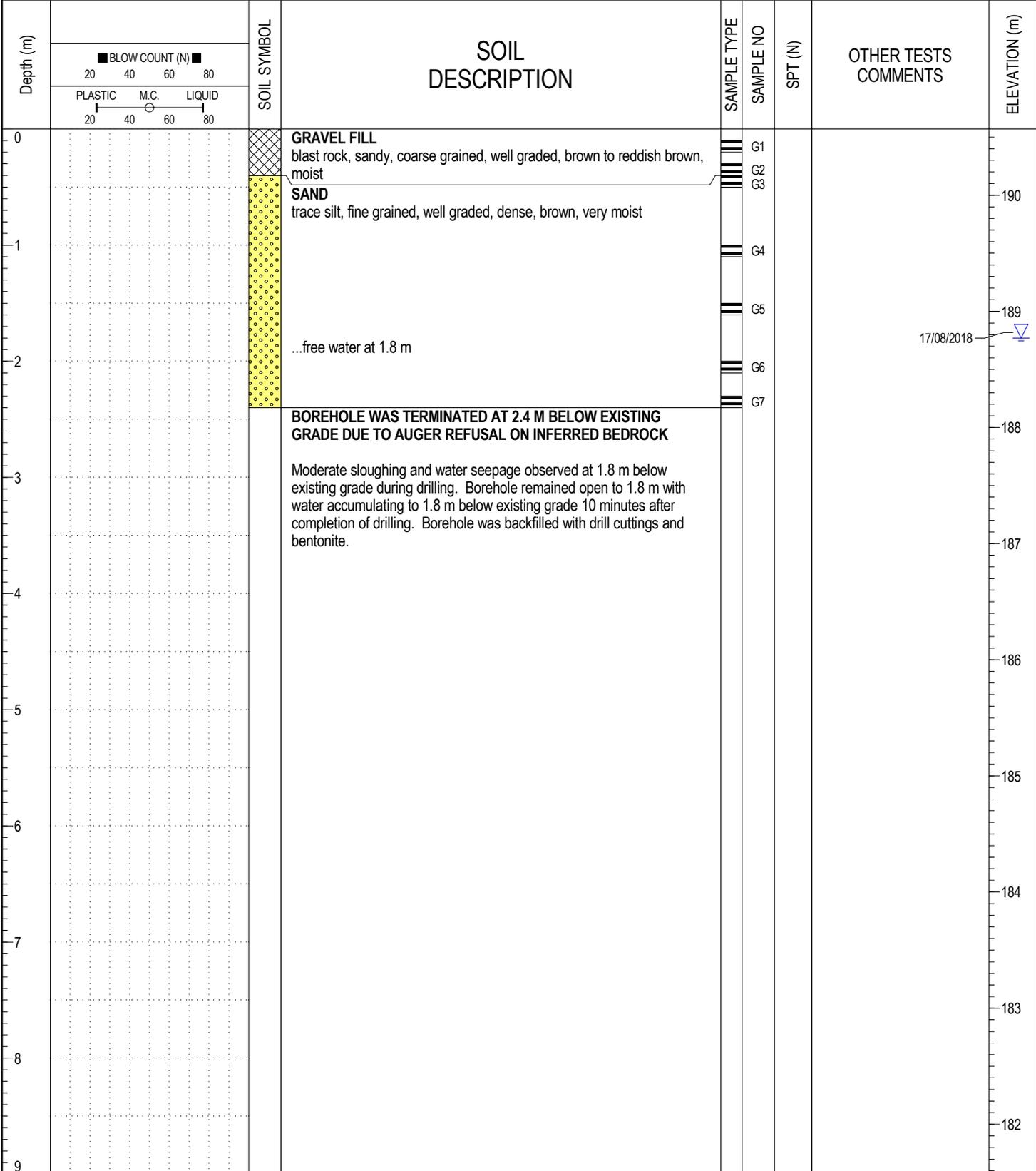


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COMPLETION DEPTH: 5.4 m  
COMPLETION DATE: 17/8/18

Government of the Northwest Territories	Ecole JH Sissons School Geotechnical Investigation & ESA	BOREHOLE NO.: <b>BH18-10</b>
Landmark Drilling Services Ltd	SITE: 5700 51a Ave, Yellowknife, NT	PROJECT NO.: YX18006
Solid Stem Augers	NAD83(CSRS) / UTM zone 11N N:6926681 E:635336	ELEVATION: 190.6 m
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BACKFILL TYPE	<input checked="" type="checkbox"/> Bentonite <input type="checkbox"/> Pea Gravel <input type="checkbox"/> Slough <input type="checkbox"/> Grout <input type="checkbox"/> Drill Cuttings <input type="checkbox"/> Sand	



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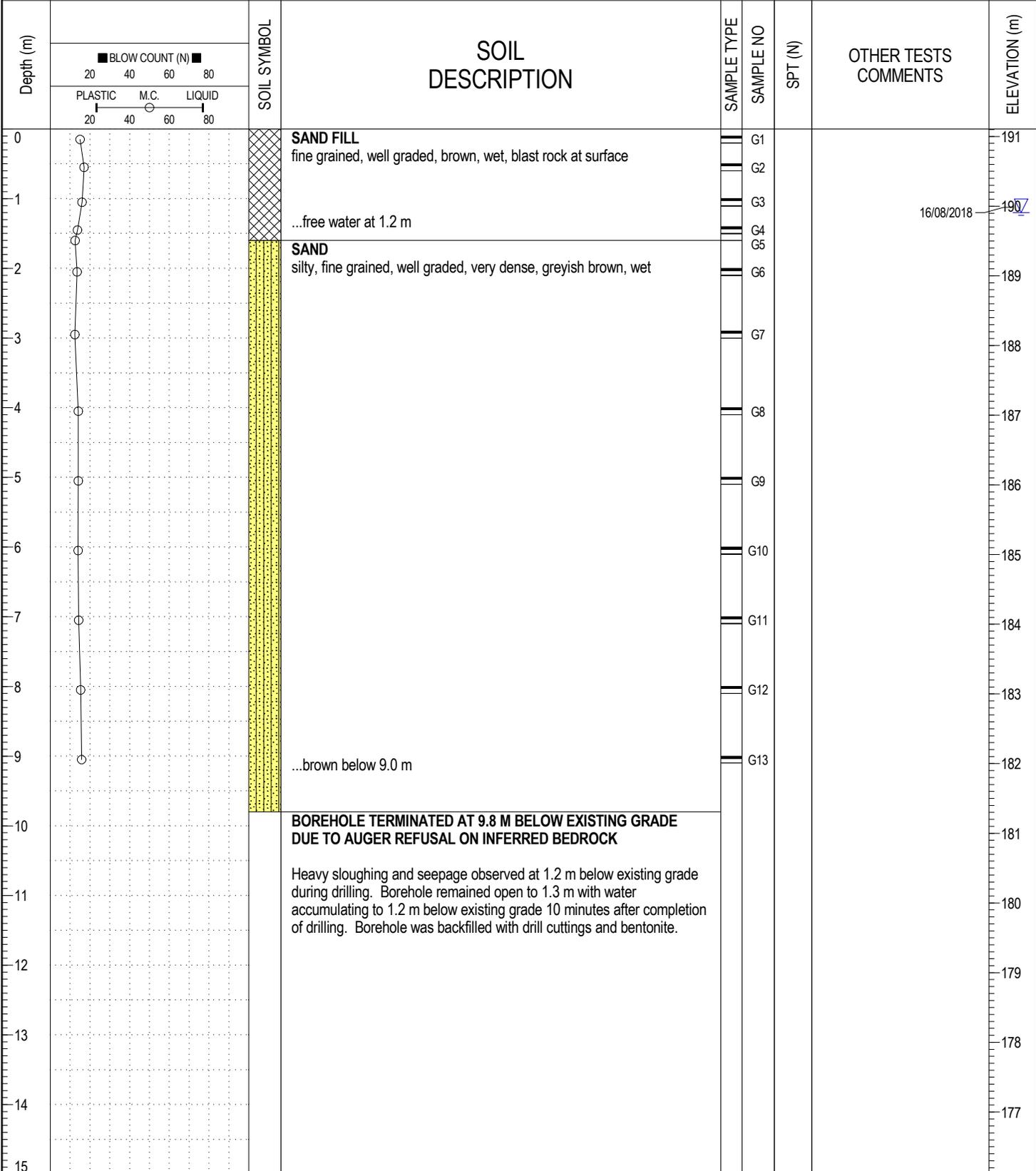


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COMPLETION DEPTH: 2.4 m  
COMPLETION DATE: 17/8/18

Government of the Northwest Territories	Ecole JH Sissons School Geotechnical Investigation & ESA	BOREHOLE NO.: <b>BH18-11</b>
Landmark Drilling Services Ltd	SITE: 5700 51a Ave, Yellowknife, NT	PROJECT NO.: YX18006
Solid Stem Augers	NAD83(CSRS) / UTM zone 11N N:6926713 E:635333	ELEVATION: 191.1 m
SAMPLE TYPE	<input checked="" type="checkbox"/> Shelby Tube <input type="checkbox"/> No Recovery <input type="checkbox"/> SPT Test (N) <input type="checkbox"/> Grab Sample <input type="checkbox"/> Split-Pen <input type="checkbox"/> Core	
BACKFILL TYPE	<input checked="" type="checkbox"/> Bentonite <input type="checkbox"/> Pea Gravel <input type="checkbox"/> Slough <input type="checkbox"/> Grout <input type="checkbox"/> Drill Cuttings <input type="checkbox"/> Sand	



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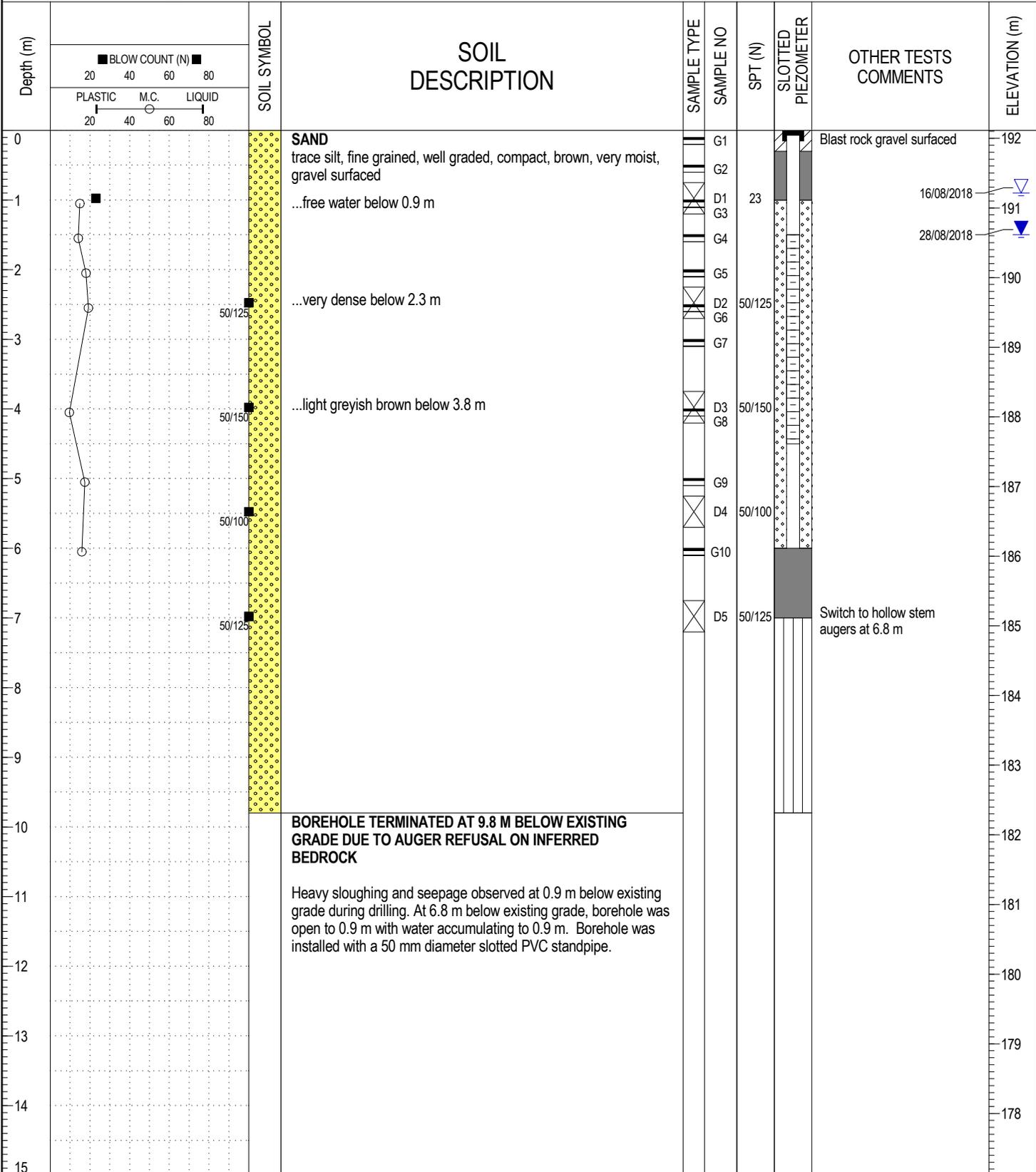
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COMPLETION DEPTH: 9.8 m  
COMPLETION DATE: 16/8/18

Government of the Northwest Territories	Ecole JH Sissons School Geotechnical Investigation & ESA	BOREHOLE NO.: <b>BH18-12</b>
Landmark Drilling Services Ltd	SITE: 5700 51a Ave, Yellowknife, NT	PROJECT NO.: YX18006
Solid & Hollow Stem Augers	NAD83(CSRS) / UTM zone 11N N:6926744 E:635328	ELEVATION: 192.1 m

SAMPLE TYPE	Shelby Tube	No Recovery	SPT Test (N)	Grab Sample	Split-Pen	Core
BACKFILL TYPE	Bentonite	Pea Gravel	Slough	Grout	Drill Cuttings	Sand



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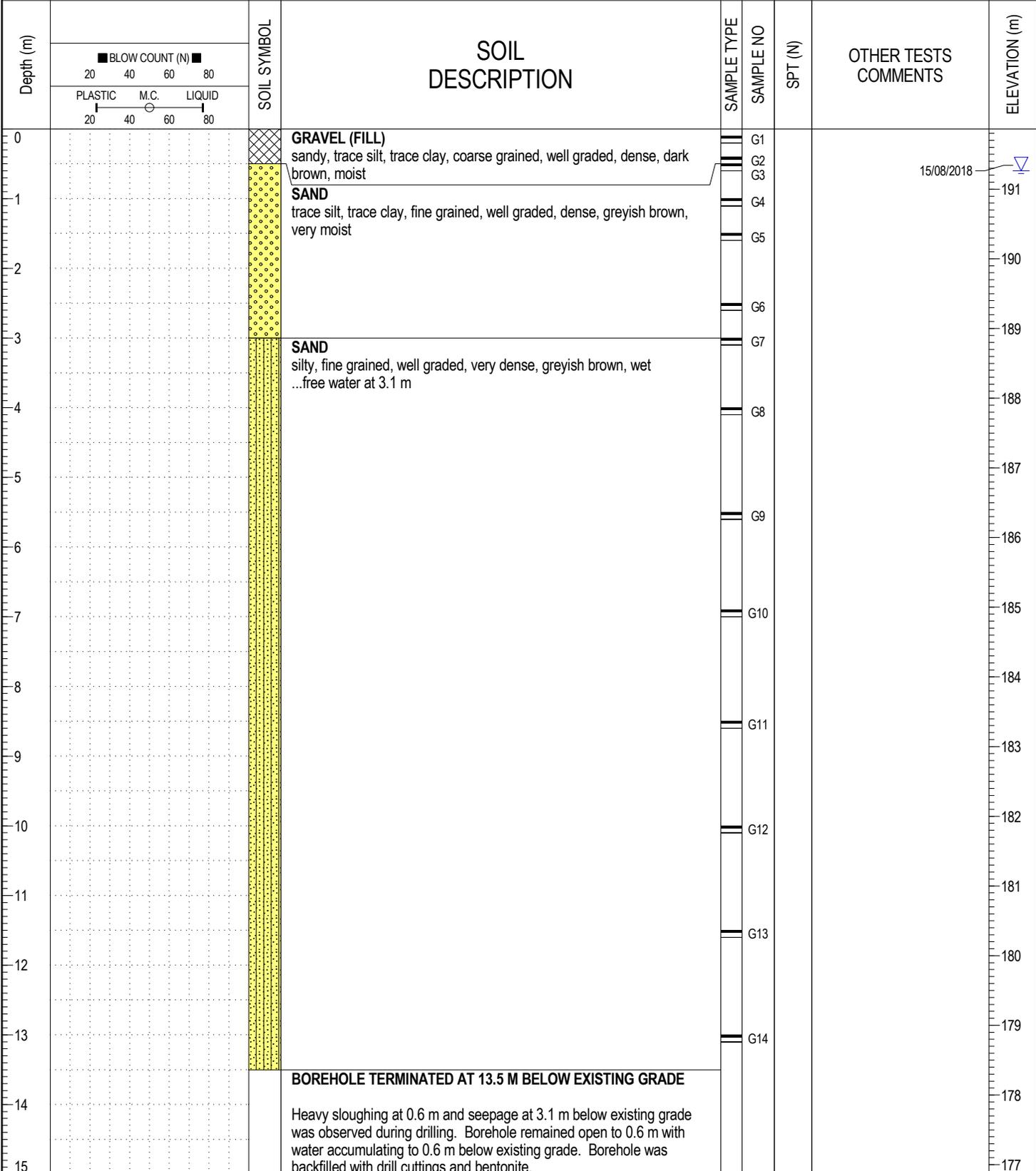


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COMPLETION DEPTH: 9.8 m  
 COMPLETION DATE: 16/8/18

Government of the Northwest Territories	Ecole JH Sissons School Geotechnical Investigation & ESA	BOREHOLE NO.: <b>BH18-13</b>
Landmark Drilling Services Ltd	SITE: 5700 51a Ave, Yellowknife, NT	PROJECT NO.: YX18006
Solid Stem Augers	NAD83(CSRS) / UTM zone 11N N:6926725 E:635311	ELEVATION: 191.9 m
SAMPLE TYPE	<input checked="" type="checkbox"/> Shelby Tube <input type="checkbox"/> No Recovery <input type="checkbox"/> SPT Test (N) <input type="checkbox"/> Grab Sample <input type="checkbox"/> Split-Pen <input type="checkbox"/> Core	
BACKFILL TYPE	<input checked="" type="checkbox"/> Bentonite <input type="checkbox"/> Pea Gravel <input type="checkbox"/> Slough <input type="checkbox"/> Grout <input type="checkbox"/> Drill Cuttings <input type="checkbox"/> Sand	



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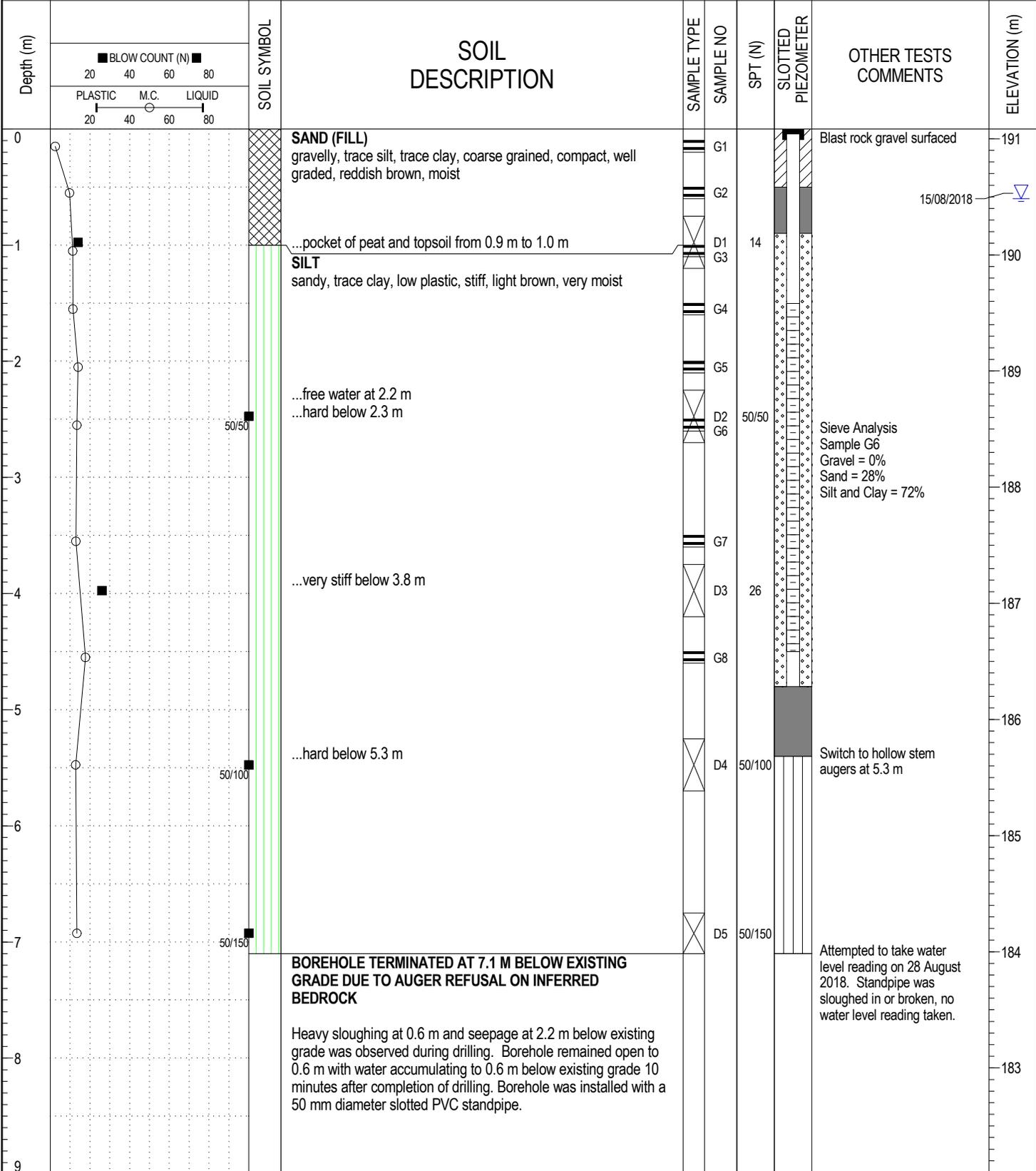


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COMPLETION DEPTH: 13.5 m  
COMPLETION DATE: 15/8/18  
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Government of the Northwest Territories	Ecole JH Sissons School Geotechnical Investigation & ESA	BOREHOLE NO.: <b>BH18-14</b>
Landmark Drilling Services Ltd	SITE: 5700 51a Ave, Yellowknife, NT	PROJECT NO.: YX18006
Solid & Hollow Stem Augers	NAD83(CSRS) / UTM zone 11N N:6926693 E:635302	ELEVATION: 191.1 m
<b>SAMPLE TYPE</b>	<input checked="" type="checkbox"/> Shelby Tube <input type="checkbox"/> No Recovery <input type="checkbox"/> SPT Test (N) <input type="checkbox"/> Grab Sample <input type="checkbox"/> Split-Pen <input type="checkbox"/> Core	
<b>BACKFILL TYPE</b>	<input checked="" type="checkbox"/> Bentonite <input type="checkbox"/> Pea Gravel <input type="checkbox"/> Slough <input type="checkbox"/> Grout <input type="checkbox"/> Drill Cuttings <input type="checkbox"/> Sand	



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## MODIFIED UNIFIED CLASSIFICATION SYSTEM FOR SOILS

MAJOR DIVISION		GROUP SYMBOL	GRAPH SYMBOL	COLOUR CODE	TYPICAL DESCRIPTION	LABORATORY CLASSIFICATION CRITERIA		
COARSE GRAINED SOILS (MORE THAN HALF BY WEIGHT LARGER THAN 75µm)	GRAVELS MORE THAN HALF THE COARSE FRACTION LARGER THAN 4.75mm	CLEAN GRAVELS (LITTLE OR NO FINES)	GW		ORANGE	WELL GRADED GRAVELS, GRAVEL-SAND MIXTURES, LITTLE OR NO FINES	$C_u = \frac{D_{60}}{D_{10}} > 4; C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}} = 1 \text{ to } 3$	
			GP		ORANGE	POORLY GRADED GRAVELS, GRAVEL-SAND MIXTURES, LITTLE OR NO FINES		NOT MEETING ABOVE REQUIREMENTS
		DIRTY GRAVELS (WITH SOME FINES)	GM		ORANGE	SILTY GRAVELS, GRAVEL-SAND-SILT MIXTURES	CONTENT OF FINES EXCEEDS 12 %	ATTERBERG LIMITS BELOW "A" LINE OR P.I. LESS THAN 4
			GC		ORANGE	CLAYEY GRAVELS, GRAVEL-SAND- CLAY MIXTURES		ATTERBERG LIMITS ABOVE "A" LINE P.I. MORE THAN 7
	SANDS MORE THAN HALF THE COARSE FRACTION SMALLER THAN 4.75mm	CLEAN SANDS (LITTLE OR NO FINES)	SW		YELLOW-BLACK	WELL GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES	$C_u = \frac{D_{60}}{D_{10}} > 6; C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}} = 1 \text{ to } 3$	
				SP		YELLOW-BLACK		POORLY GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES
		DIRTY SANDS (WITH SOME FINES)	SM		YELLOW-BLACK	SILTY SANDS, SAND-SILT MIXTURES	CONTENT OF FINES EXCEEDS 12 %	ATTERBERG LIMITS BELOW "A" LINE OR P.I. LESS THAN 4
			SC		YELLOW-BLACK	CLAYEY SANDS, SAND-CLAY MIXTURES		ATTERBERG LIMITS ABOVE "A" LINE P.I. MORE THAN 7
	FINE-GRAINED SOILS (MORE THAN HALF BY WEIGHT SMALLER THAN 75µm)	SILTS BELOW "A" LINE NEGLECTIBLE ORGANIC CONTENT	$w_L < 50\%$	ML		GREEN	INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTY SANDS OF SLIGHT COMPRESSIBILITY	CLASSIFICATION IS BASED UPON PLASTICITY CHART (SEE BELOW)  $I_p = w_L - w_p$ $I_L = \frac{w - w_p}{w_L - w_p} = \frac{w - w_p}{I_p}$ $I_p = \text{PLASTICITY INDEX}$ $I_L = \text{LIQUIDITY INDEX}$
			$w_L > 50\%$	MH		BLUE	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS, FINE SANDS OR SILTY SOILS OF HIGH COMPRESSIBILITY	
CLAYS ABOVE "A" LINE NEGLECTIBLE ORGANIC CONTENT		$w_L < 30\%$	CL		GREEN	INORGANIC CLAYS OF LOW PLASTICITY, GRAVELLY, SANDY OR SILTY CLAYS, LEAN CLAYS		
		$30\% < w_L < 50\%$	CI		GREEN-BLUE	INORGANIC CLAYS OF MEDIUM PLASTICITY, SILTY CLAYS		
		$w_L > 50\%$	CH		BLUE	INORGANIC CLAYS OF HIGH PLASTICITY, FAT CLAYS		
ORGANIC SILTS & CLAYS BELOW "A" LINE		$w_L < 50\%$	OL		GREEN	ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY	WHENEVER THE NATURE OF THE FINES CONTENT HAS NOT BEEN DETERMINED, IT IS DESIGNATED BY THE LETTER "F". E.G. SF IS A MIXTURE OF SAND WITH SILT OR CLAY	
		$w_L > 50\%$	OH		BLUE	ORGANIC CLAYS OF HIGH PLASTICITY		
HIGHLY ORGANIC SOILS		Pt		PURPLE	PEAT AND OTHER HIGHLY ORGANIC SOILS	STRONG COLOUR OR ODOUR, AND OFTEN FIBROUS TEXTURE		

### SPECIAL SYMBOLS

LIMESTONE		LEAN OIL SAND / RICH OIL SAND	
SANDSTONE		SHALE	
SILTSTONE		FILL (UNDIFFERENTIATED)	

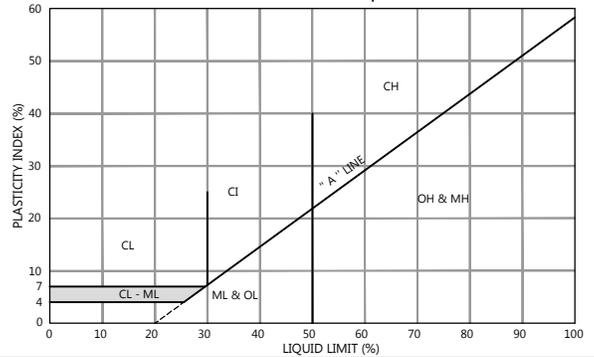
### SOIL COMPONENTS

FRACTION	U.S. STANDARD SIEVE SIZE		DEFINING RANGES OF PERCENTAGE BY WEIGHT OF MINOR COMPONENTS	
	PASSING	RETAINED	PERCENT	DESCRIPTOR
GRAVEL	76mm	19mm	35-50	AND
	19mm	4.75mm		
SAND	4.75mm	2.00mm	20-35	Y/EY
	2.00mm	425µm	10-20	SOME
	425µm	75µm	1-10	TRACE
FINES (SILT OR CLAY BASED ON PLASTICITY)	75µm			

### OVERSIZED MATERIAL

ROUNDED OR SUBROUNDED: COBBLES 76mm TO 200mm BOULDERS > 200mm	NOT ROUNDED: ROCK FRAGMENTS > 76mm ROCKS > 0.76 CUBIC METRE IN VOLUME
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PLASTICITY CHART FOR SOILS PASSING 425 µm SIEVE



**NOTES:**

- ALL SIEVE SIZES MENTIONED ON THIS CHART ARE U.S. STANDARD A.S.T.M. E.11
- COARSE GRAIN SOILS WITH 5 TO 12% FINES GIVEN COMBINED GROUP SYMBOLS, E.G. GW-GC IS A WELL GRADED GRAVEL SAND MIXTURE WITH CLAY BINDER BETWEEN 5 AND 12% FINES.



Environment & Infrastructure Solutions  
 5681-70 STREET, EDMONTON, ALBERTA, T6B 3P6  
 PHONE 780-436-2152, FAX 780-435-8425