GEOTECHNICAL SITE INVESTIGATION
Elk Island Public Schools Davidson Creek New Elementary School

Davidson Creek Park
Davenport Drive & Davenport Place
Sherwood Park, AB

Prepared for: Elk Island Public Schools
Prepared by: Opus Stewart Weir Ltd.
Our File No.: S-38765.00
Date: May 18, 2016

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Date: May 18, 2016

Elk Island Public Schools

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Attention: **Robert Graham, P.Eng.**
Assistant Director – Facility Services

Re: Geotechnical Site Investigation
Proposed Davidson Creek New Elementary School
Davidson Creek Park, Davenport Drive & Davenport Place
Sherwood Park, AB

We have enclosed with this letter our geotechnical site investigation report for the above referenced project. The geotechnical investigation was conducted by drilling boreholes, and performing in-situ field tests and laboratory tests on selected soil samples. The report provides geotechnical recommendations for the design and construction of the proposed elementary school building.

We would like to thank you for the opportunity to provide our services on this project. If you have questions or require additional information, please contact our office.

Yours truly,
Opus Stewart Weir Ltd.
APEGA Permit No.: **P 292**

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1 INTRODUCTION

This report presents results of a geotechnical site investigation and recommendations for the proposed elementary school located at the Davidson Creek Park in Sherwood Park, AB. Opus Stewart Weir Ltd. (OSW) has prepared this report for Elk Island Public Schools.

2 SITE AND PROJECT DESCRIPTION

The project site is located within the Davidson Creek Park in Sherwood Park, AB. The site is bounded by Davenport Drive at the north and Davenport Place at the east. The project site is currently serving as a baseball field (or diamond). It has near level topography with drainage swales surrounding the diamond. A catch basin also exists at the northeast of the ball diamond. Representative site pictures taken during field drilling are attached in Appendix A.

It is understood that the development will consist of a new K-6 elementary school building. The proposed building may have two stories. The development will also consist of a parking area in the middle of the lot, and bus and drop-off lanes adjacent to Davenport Drive and Davenport Place. No other information was provided regarding the proposed project during the preparation of this report.

3 SCOPE OF WORK

The scope of work for this geotechnical site investigation includes:

- Determining subsurface soil profiles and their geotechnical characteristics;
- Determining groundwater and sloughing conditions;
- Comments on site preparation
- Comments on frost heave and soil swelling;
- Recommendations of foundation types and soil design parameters;
- Recommendations for design and construction of grade-supported slabs;
- Recommendations of soil design parameters for basement walls;
- Comments on surface and subsurface drainage;
- Recommendations of asphalt pavement structure for at-grade parking;
- Comments on sulphate attacks on concrete; and
- Seismic site classification in accordance to Table 4.1.8.4.A of the 2005 National Building Code of Canada.

4 METHOD OF INVESTIGATION

4.1 Field Investigation

Nine geotechnical boreholes (BH16-01 to BH16-09) were drilled on May 02, 2016 at the project site. The depths and locations of the boreholes are summarized in Table 1. The borehole locations are also shown in a site plan attached to Appendix B.

TABLE 1: BOREHOLE LOCATIONS AND DEPTHS

Boreholes	UTM Coordinate (12U)		Depth (m)	Locations
	North	East		
BH16-01	5936008	350248	9.47	Within proposed building footprint
BH16-02	5936061	350227	8.71	Within proposed building footprint
BH16-03	5936020	350297	8.71	Within proposed building footprint
BH16-04	5935956	350285	8.71	Within proposed building footprint
BH16-05	5936059	350263	4.5	Within proposed park area
BH16-06	5935985	350306	4.5	Within proposed park area
BH16-07	5935961	350242	4.5	Within proposed park area
BH16-08	5936023	350195	4.5	Within proposed park area
BH16-09	5936065	350190	4.5	Within proposed park area

Field drilling was carried out using a track drill rig owned and operated by **All Service Drilling Inc.** equipped with 150 mm diameter solid stem augers and SPT capability. The soil sampling and logging of various soil strata was performed by Dale Johnston, Geotechnical Project Manager from our Sherwood Park office. The soil conditions encountered during drilling were described visually in accordance with the Modified Unified Soil Classification System (MUSCS). Disturbed samples were retrieved from solid-stem augers at 0.75 m depth intervals, and from SPT split spoon sampler. Field SPT tests were conducted at depth intervals of 1.5 m in boreholes BH16-01 to BH16-04. Pocket Penetrometer readings were also taken at depth intervals of 0.75 m on cohesive soils in all boreholes. Standpipes were installed in boreholes BH16-01 to BH16-04 to monitor the long-term groundwater levels. The borehole logs are attached in Appendix B.

4.2 Laboratory Testing

Laboratory testing was carried out on selected soil samples which included:

- In-situ moisture content tests
- Atterberg limit tests
- Sulphate tests

The moisture content tests were conducted on all retrieved samples at the depth interval of 0.75 m. The locations of the remaining laboratory tests are summarized in Table 2.

The laboratory test results are presented in Appendix C. They are also summarized in the borehole logs attached in Appendix B.

TABLE 2: LOCATIONS OF LABORATORY TESTS

Atterberg Limit Tests			
Borehole #	**Depth (m)**	**Borehole #**	**Depth (m)**
BH16-02	4.5	BH16-07	2.25
BH16-03	2.25	BH16-09	3.0
Sulphate Tests			
Borehole #	**Depth (m)**	**Borehole #**	**Depth (m)**
BH16-01	3.0	BH16-04	1.5
BH16-02	4.5	-	-

5 SUBSURFACE CONDITIONS

5.1 Subsurface Soil Profiles

The subsurface soil profile encountered in the boreholes generally consisted of topsoil at the surface underlain by clay till deposit. Detailed descriptions of the soil strata are provided in the following sub-sections.

5.1.1 Topsoil

Topsoil was encountered at the surface of all the boreholes. It had thicknesses ranging from 150 mm to 250 mm. It was described as black and damp.

5.1.2 Clay Till

Clay till deposit was encountered below the topsoil and extended to the termination depths of all boreholes. It was generally described as silty, some sand to sandy, brown to grey, moist, stiff, and occasional gravel. Laboratory and field test results on selected clay till soil samples are summarized in Table 3.

TABLE 3: PROPERTIES OF CLAY TILL

Test	Values
Pocket Penetrometer (kPa)	100 to 250
Standard Penetration Test (SPT), blow Counts	8 to 16
Natural Moisture Content, W_n (%)	9 to 23
Liquid Limit (%)	35 to 40
Plastic Limit (%)	13 to 14
USCS Classification	CI

5.2 Soil Sloughing and Groundwater Condition

The soil sloughing and groundwater condition in the boreholes were observed during drilling. Groundwater levels were also measured on May 13, 2016 from the installed standpipes in boreholes BH16-01 to BH16-04. The summary of the observations are presented in Table 4.

TABLE 4: SOIL SLOUGHING AND GROUNDWATER LEVELS

Borehole Number	Depths Below Ground Surface (m)			
	Sloughing Depth	Seepage During Drilling	Groundwater Level	
			At Completion	On May 13, 2016
BH16-01	No	9.0	No	4.74
BH16-02	No	8.6	No	4.5
BH16-03	8.25	6.2	6.6	3.35
BH16-04	8.25	5.4	7.3	3.25
BH16-05 to BH16-09	No	No	No	No standpipe

It should be noted that the observed groundwater levels were for short term and may not represent long term stabilized groundwater levels, which will vary seasonally. The actual groundwater conditions at the time of construction could vary from those recorded during this investigation, and should be monitored during construction.

5.3 Water Soluble Sulphate Content Test Results

The sulphate content tests were conducted on selected soil samples at locations listed in Table 2. The test results are summarized in Table 5.

TABLE 5: SULPHATE CONTENT TEST RESULTS

Locations	BH16-01 @ 3.0 m	BH16-02 @ 4.5 m	BH16-04 @ 1.5 m
Contents (%)	0.1	0.06	0.09

All the soluble sulphate test results revealed a “negligible to moderate” potential for sulphate attack on concrete in contact with native soils.

All concrete in contact with the native soils at this site should be made from CSA Type 50 (HS) sulphate resistant cement possessing a minimum 56 day compressive strength of 30 MPa. The maximum w/c ratio should be 0.5. An air entrainment agent of 5% to 7% is recommended for improving workability and durability of concrete.

6 DISCUSSIONS AND RECOMMENDATIONS

6.1 General

This section of the report consists of discussions and recommendations regarding the geotechnical aspects for the design and construction of the proposed school building and asphalt parking. They are provided based on the subsurface information obtained from the geotechnical investigation at the subject site, as presented in the previous sections.

Where the subsurface conditions encountered during construction are different from those stated in the previous sections of this report, Opus Stewart Weir Ltd. should be provided with the opportunity to revise the geotechnical comments and recommendations contained in this report.

6.2 Site Preparation

6.2.1 Removal of Unsuitable Materials

Topsoil was encountered at the surface of the boreholes drilling across the project site. The thickness of the topsoil ranged from 150 mm to 250 mm.

All topsoil, organics, uncontrolled fills, and other unsuitable materials should be subexcavated and removed from the footprint of the proposed school building and parking area.

6.2.2 Site Grading

The discussions and recommendations contained in this report are provided based on the assumption of no significant site grade raise from the original ground level (OGL). If significant site grade raise is considered, the comments and recommendations in this report should be revised.

The general site grading is recommended to promote positive drainage of surface water away from the footprints of the proposed building and parking area.

6.2.3 Excavations

All excavations should be properly designed and conducted by experienced contractors. The effects of construction equipment and stockpiling of excavated soils at the crest of excavations should be considered during the design of excavations. Opus Stewart Weir Ltd. can also design excavations at the time of construction.

As a minimum requirement, Part 32 and other applicable sections of the *Alberta Occupational Health and Safety Regulations (AOHSR)* shall be followed for temporary excavations.

Based on the measured groundwater levels during field investigation (see Table 4), it is our opinion that shallow groundwater levels may be encountered at the project site at the time of construction. Temporary dewatering systems may be required during excavation.

Care should be taken to avoid the exposed subgrade after excavation from becoming disturbed or frozen. Water should not be allowed to pond directly on exposed subgrade soils as it can potentially soften the soil and reduce its bearing capacity. Construction traffic (human and equipment) should also be minimized on the exposed subgrade to reduce disturbance. Exposed subgrade soils can be protected from disturbance using lean concrete or gravel. Site specific recommendations for protecting bearing subgrade from disturbance or freezing can be provided by Opus Stewart Weir Ltd. at the time of construction.

After any excavation work, the exposed subgrade should be reviewed and approved by Opus Stewart Weir Ltd. prior to the placement of engineered fill.

6.2.4 Engineered Fill

Structural engineered fill required to bring the approved subgrade to design grade within the footprint of the proposed building is recommended to consist of 80 mm minus pitrun gravel or 25 mm minus crushed gravel. It can also consist of low to medium plastic clay (or till) fill. However, larger settlement (about 1% of fill thickness) is expected for clay engineered fill compared to granular engineered fill (about 0.5% of fill thickness). The engineered fill material must be free of any organic materials, contamination, debris, and other deleterious materials. The structural engineered fill within the footprints of the proposed structures should be compacted to 100% of Standard Proctor Maximum Dry Density (SPMDD) within ±2% of its Optimum Moisture Content (OMC) in lifts of 150 mm. The fill placement and compaction should extend beyond the footprints to at least 1 m or thickness of fill whichever is greater.

The native clay till soil may be considered for general engineered fill outside the footprints of the proposed school building such as for subgrade fill at parking areas, access roads, utility trenches, and for general site grading. Estimated optimum moisture contents of the native clay till soil are presented in the Lab Summary Sheet in Appendix C. The native soils should be moisture conditioned and compacted to the minimum of 98% SPMDD at ±2% OMC in lifts of 150 mm. Imported low to medium plastic clay soil can be used for additional general engineered fill.

Engineered fill should not be placed and compacted over frozen soil, and not frozen when placed. The specification, placement, and compaction of engineered fills should be reviewed and approved by a qualified geotechnical engineer.

6.3 Frost Heave and Soil Swelling

6.3.1 Frost Heave

6.3.1.1 Frost Penetration Depth

The estimated maximum depth of frost penetration at the project site is **2.4 m**. The frost penetration analysis is based on a freezing index for a 30-year return period of 1,450 degree-days Celsius. The analysis is also based on the assumption of a clay till soil type without surface cover.

6.3.1.2 Frost Protection

Unheated footings are recommended to extend below the frost penetration depth to provide adequate frost cover. Partially heated exterior footings are also recommended to extend to a minimum depth of 1.5 m. The footing depths can be reduced with the provision of rigid heat insulation for the footings. Unheated or partially heated grade supported slabs should also be insulated with rigid insulations.

The type of insulation must be selected to resist chemical or hydrocarbon attack expected at the project site. The insulation must also be suitable for below grade use, and must have sufficient compressive strength to support the anticipated traffic and/or design loads.

Grade beams and pile caps must be provided with adequate void spaces or compressible materials between their undersides and the soil, as discussed in detail in Section 6.4.3 of this report.

Unheated or partially heated pile foundations must be designed for adfreeze uplift forces. Details of the design considerations are provided in the Pile Foundation section of this report, Section 6.4.2.

6.3.2 Soil Swelling

The native clay till soil is classified as medium plastic based on the test results on selected soil samples presented in Table 3. Its liquid limit ranged from 35% to 40% while its plastic limit ranged from 13% to 14%. The test results also indicated that the natural moisture contents of the native clay till soil ranged from 9% to 23% with average value of 15%. The medium plastic clay till soil may undergo swelling and exert low to moderate pressure on structures upon the absorption of additional moisture. Footings, slab-on-grades, grade beams, pile caps, and sidewalks may be affected by the soil swelling.

The cost effective method for the protection of soil swelling is by controlling the moisture of the native clay till soil, under the footprints of the structures, so that it will not vary from its natural moisture content value. This can be achieved by providing an effective surface drainage system to reduce infiltration of surface water to the clay till subgrade soil. In addition, the native clay till soil must not be subjected to excessive drying during excavation. In addition to moisture controlling, the grade beams and pile caps must be provided with adequate void spaces or compressible materials between their undersides and the native clay till soil.

6.4 Foundations

6.4.1 Strip and Square Footings

Footing foundations placed on the native clay till soil may be considered for the proposed school building. The recommended bearing capacities for footing designs are provided in Table 6.

The footings should be placed on the suitable, native, stiff clay till bearing soil. The depths of the footings should be determined based on the frost penetration cover requirement as stated in Section 6.3.1.2.

TABLE 6: BEARING CAPACITIES FOR FOOTINGS

Footing Type	Ultimate Bearing Resistance (kPa)	Factored Ultimate Bearing Resistance* (kPa)
Square	360	180
Strip	300	150

*The factored ultimate bearing resistance was determined using a geotechnical resistance factor of 0.5.

Based on the measured groundwater levels during field investigation (see Table 4), it is our opinion that shallow groundwater levels may be encountered at the project site at the time of basement excavation. Temporary dewatering of excavations may be required for placement of footings.

The footings should not be placed on disturbed or frozen bearing soil. Foundation excavation must be protected from frost, desiccation, ingress of water, or from disturbance due to construction traffic. Bearing soils which become frozen, dried, or softened should be removed and replaced with lean concrete.

All bearing soils should be reviewed by a qualified geotechnical engineer prior to the placement of footings.

6.4.2 Cast-In-Place Concrete Piles

Pile foundations can also be considered for the proposed school building. The skin friction and end bearing resistances for the design of cast-in-place piles are provided in Table 7 and Table 8, respectively.

End bearing resistances should only be considered in the design where the condition of the base of the drilled holes can be verified. The bases of all end bearing piles must be thoroughly cleaned of all loosened material. Following drilling and cleaning, pile bores should be inspected by a qualified geotechnical engineer to confirm an adequate bearing surface is prepared at an appropriate depth.

TABLE 7: SKIN FRICTION RESISTANCES FOR CONCRETE PILES

Soil Type	Depth (mbgs*)	Ultimate Skin Friction (kPa)	Factored Skin Friction (kPa)	
			Compression***	Uplift****
Clay Till	0.0 – 1.5 (3.0**)	0	0	0
	1.5 (3.0**) – 9.0	40	16	12

*mbgs stands for meter below existing ground surface
**3.0 m stands for basement level
***Factored ultimate skin friction resistances for compressive loading were calculated using a geotechnical resistance factor of 0.4.
****Factored ultimate skin friction resistances for uplift loading were calculated using a geotechnical resistance factor of 0.3.

TABLE 8: END BEARING RESISTANCES FOR CONCRETE PILES

Soil Type	Depth (mbgs*)	Pile Base Diameter (m)	Ultimate End Bearing Resistance (kPa)	Factored Ultimate End Bearing Resistance (kPa)**
Clay Till	0.0 – 6.0	N/A	0	0
	6.0 – 9.0	D < 0.5	575	230
		0.5 < D < 1.0	450	180
		D > 1.0	375	150

*mbgs stands for meter below existing ground surface
**Factored ultimate skin friction resistances for compressive loading were calculated using a geotechnical resistance factor of 0.4.

The presence of groundwater may cause construction difficulties of cast-in-place piles due to soil sloughing and water seepage. The use of casing is required in order to reduce the soil sloughing and water seepage. In addition to the use of casing, concrete should be poured as soon as practical in order to reduce the amount of water seepage.

Piles subjected to frost heave should be evaluated for adequate uplift resistance using an average adfreeze bond stress of 65 kPa. The estimated depth of frost penetration is 2.4 m as determined in Section 6.3 of this report. The frost jacking force should be balanced by the dead load acting on the pile, the weight of the pile, and the skin friction resistance below the frost zone in order to determine the required embedment length of the pile. Since the adfreeze values are ultimate, the ultimate skin friction values can be used.

As a minimum requirement, piles should be embedded to a minimum depth of 6.0 m below the ground surface. A minimum pile shaft diameter of 400 mm is recommended to minimize void formation during pouring of the concrete.

The installation of concrete piles should be inspected fulltime by a qualified geotechnical engineer or technician. The inspector should document complete and accurate records of the pile installation operation.

6.4.3 Pile Caps and Grade Beams

Precautions should be taken to minimize the potential of heaving of pile caps and grade beams due to frost penetration or swelling of the underlying soil. The heaving forces can be greatly reduced by the placement of a compressible material or by providing a void space using a void form product, between the underside of the structure and the soil. The minimum thickness of void space should be 150 mm. If a compressible material is used as an alternative to void form product, its thickness should be properly designed. The backfills around the pile caps and grade beams should consist of non-frost susceptible, low plastic materials. The finished grade adjacent to pile caps and grade beams should be capped with low plastic clay and sloped away so that the surface runoff is not accumulated in the void space or in the compressible medium.

6.5 Grade Supported Concrete Slabs

6.5.1 Subgrade Preparation

In general, the subgrade for placement of concrete slabs should be prepared considering Section 6.2.

After excavation to the design grade slab level, the exposed subgrade condition should be visually inspected by a qualified geotechnical engineer or technician. As per the boreholes, a firm to stiff clay till subgrade is expected at the proposed buildings footprint. Local soft subgrade areas should be sub-excavated and replaced with engineered fill or lean concrete. The exposed subgrade should also be protected from disturbance by construction traffic using gravel or lean concrete. The prepared subgrade should be reviewed by a qualified geotechnical engineer or technician.

6.5.2 Granular Bedding

A minimum of 200 mm thick layer of 20 mm minus well-graded crushed gravel with less than 5% of fine content is recommended as a bedding layer underneath floor slabs for the purpose of levelling and drainage. The crushed gravel should be placed in one layer and compacted to 100% of its SPMDD at ±2% OMC.

It is recommended to design grade supported slabs as floating slabs independent of foundations and grade beams in order to reduce the effect of differential movements, if any, between slabs and other structural components. Unheated grade supported slabs should be insulated in order to reduce the frost heave.

6.6 Basement Walls

6.6.1 Lateral Earth Pressure Coefficients

The recommended lateral earth pressure coefficients for the calculation of lateral earth pressures acting against the basement walls are provided in Table 9.

TABLE 9: LATERAL EARTH PRESSURE COEFFICIENTS

Soil Type	Ka	Kp	Ko
Clay Till	0.44	2.27	0.61

6.6.2 Backfill

The backfill against the basement walls can consist of the native clay till or imported low to medium plastic clay with a drainage board of J-DRAIN 200 or equivalent installed at the back of concrete walls. Alternately, the backfill can consist of free-draining, clean, granular material. The granular backfill is recommended to be wrapped up with nonwoven geotextile (NILEX 4553 or equivalent). The granular backfill should be capped at the top with 500 mm thick low plastic clay to seal the ingress of runoff water towards the backfill.

The backfills behind the walls should be compacted moderately to 95 % of SPMDD at its OMC in horizontal lifts of 200 mm. Higher compaction should be avoided to prevent excessive pressures that may damage the wall.

Concrete walls should be protected with damp/water proofs.

6.7 Drainage

6.7.1 Surface Drainage

The finished grade around the proposed building should be laid out such that the surface water drains away from the building as quickly as possible. In unpaved areas, the upper 0.5 m of backfill around the building should consist of compacted low to medium plastic clay to act as a seal against the ingress of runoff water. The low to medium plastic clay should extend to a distance of 3 m away from the building periphery and be graded to a designed grade slope.

The surface drainage system should be maintained periodically. Any cracks around the proposed building should be sealed periodically to maintain watertight surface drainage. The effects of frost heave or high plastic soil swell on the footings, grade slabs and sidewalks can be reduced by controlling the moisture infiltration with the provision of an effective surface drainage system.

6.7.2 Subsurface Drainage

An exterior weeping tile system is recommended around the bottom of the basement walls or foundations in order to collect drained water behind basement walls and connect to a sump. The weeping tiles should consist of perforated rigid plastic pipes surrounded by free draining gravel filter, and both pipes and filters should be enveloped with nonwoven geotextile. The pipes should drain freely into the sump, from which water should be pumped out away from the building.

In case of groundwater level within 1 m of the proposed basement floor level, an interior weeping tile system is also recommended below the basement floor slab. The internal weeping tile system should consist of a series of perforated pipes placed parallel at a designed spacing.

6.8 Asphalt Parking

6.8.1 Subgrade Preparation

In general, the subgrade for placement of pavement structure should be prepared considering Section 6.2.

After the removal of unsuitable materials, the exposed subgrade condition should be visually inspected and proof rolled. As per the boreholes, a firm to stiff clay till subgrade is expected at the proposed parking area. Local soft subgrade areas should be sub-excavated and replaced with low plastic clay engineered fill. The exposed subgrade should also be protected from disturbance by construction traffic.

The approved exposed subgrade should be raised to the design parking subgrade level with the use of low plastic clay engineered fill. For areas where no grade raise is required, a minimum of

150 mm of the exposed subgrade should be scarified and re-compacted to a minimum of 98% of SPMDD at ±2% OMC. The prepared subgrade should be reviewed by a qualified geotechnical engineer or technician.

6.8.2 Design Inputs

Based on the subgrade preparation recommended in Section 6.8.1, a subgrade CBR of 3% is assumed for the pavement design. Based on our understanding of the project, the following traffic composition is assumed to design the flexible pavement parking:

- passenger cars
- light-weight single unit trucks (Class 3 or less, based on FHWA vehicle classification)
- limited numbers (< 25 per day) of buses and medium-weight single unit trucks (Class 4, 5 & 6, based on FHWA vehicle classification)

Based on the above assumed traffic composition, the following design 80kN ESAL (equivalent single axle load) are estimated for design life of 20 years:

- 200,000 for **light traffic areas** such as for parking stalls and driving lanes between parking stalls
- 300,000 for **heavy traffic areas** such as entrances, exits, and main driving lanes

6.8.3 Pavement Section

The flexible pavement design was performed based on AASHTO (1993) method. The recommended pavement section is presented in Table 10.

TABLE 10: FLEXIBLE PAVEMENT SECTION

Layer	Thickness (mm)*	
	Light Traffic Areas	Heavy Traffic Areas
Asphalt Concrete Pavement (ACP)	90	100
Granular Base Course (GBC)	250	275
Subgrade	Prepared as recommended	

*The pavement design should be revised if the expected traffic is higher than estimated in Section 6.8.2.

The design pavement thickness provided in Table 10 should be compared with the Strathcona County specifications for minimum pavement thickness.

6.8.4 Material and Construction Specifications

The asphalt and aggregate materials, their testing, and construction of the proposed flexible pavement should consider the Strathcona County specifications.

The structure will require proper maintenance on a periodic basis in order to provide a serviceable driving surface over the life of the structure.

6.9 Seismic Site Classification

The project site is classified for seismic response in accordance to seismic site classification of the National Building Code of Canada (2005), Table 4.1.8.4 A. The NBCC seismic site classification is based on the average values of shear wave velocities, standard penetration tests, and undrained shear strengths for the top 30 m soil deposit. The subsurface soil deposit at the project site was mainly composed of clay till deposit to the termination depth of the boreholes (i.e. 9 m). The average value of SPT blow counts for the top 9 m depth of soil deposit was about **12**.

Based on the average SPT value and Table 4.1.8.4.A of NBCC (2005), the project site is classified as **Site Class "E".**

7 FIELD REVIEW

The geotechnical discussions and recommendations contained in this report are based on the subsurface information obtained from the geotechnical boreholes and field tests conducted at representative locations within the project site. However, it should be noted that the subsurface soil condition may change between the boreholes and field test locations.

Should the information presented in this report be used for the proposed site development purposes, we recommend that on-site field reviews be performed by Opus Stewart Weir Ltd. to verify that actual site conditions are consistent with the assumed conditions taken in the design of foundations as per this report.

Based on the Alberta Building Code (ABC), adequate levels of field reviews are required for foundation construction and earthworks. The field reviews are recommended to include:

- *Field Inspections* – actual subsurface conditions exposed after excavations, final subgrade or bearing soil conditions, fulltime pile constructions, earthworks, etc.; and
- *QA/QC tests* – such as compaction tests, pile PDA tests, etc.

The field reviews are recommended to be conducted under the control of the Geotechnical Engineer of Record, as required by the ABC.

Where the subsurface conditions encountered during construction is different from stated in the this report, Opus Stewart Weir Ltd. should be provided with the opportunity to revise the geotechnical comments and recommendations contained in this report.

8 CLOSURE

This report was prepared for the exclusive use of **Elk Island Public Schools** and is the authorized user for the specific application to the project described in this report. It has been prepared in accordance with generally accepted soil and foundation engineering practice. No other warranty, expressed or implied, is made.

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APPENDIX A

Representative Site Pictures

OPUS

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NORTH OF BH16-01

Figure 001

S-38765-00

OPUS

#140, 2121 Premier Way
Sherwood Park, Alberta
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SOUTHWEST OF BH16-01

Figure 002

S-38765-00

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AT BH16-03

Figure 003

S-38765-00

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#140, 2121 Premier Way
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AT BH16-04

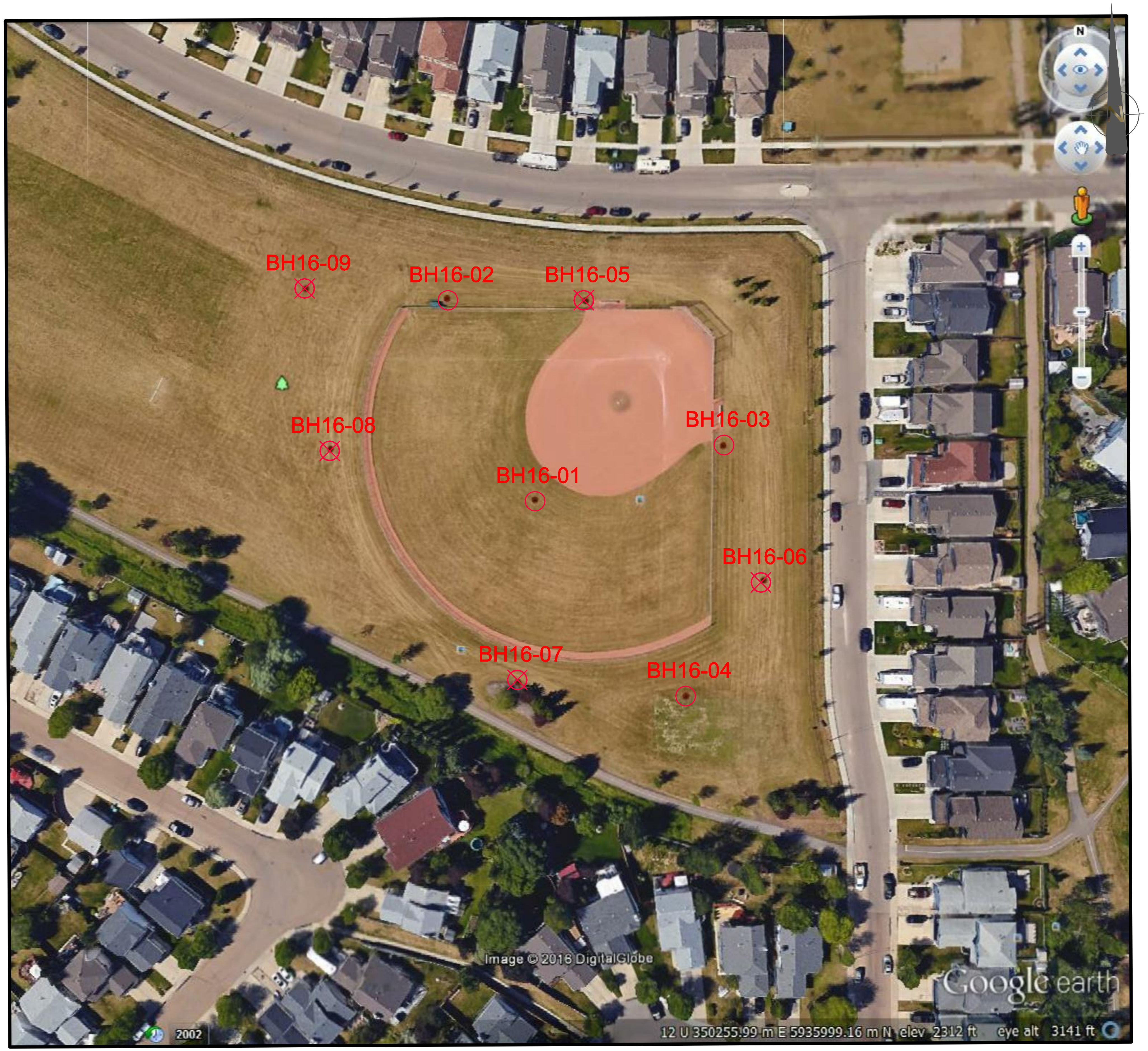
Figure 004

S-38765-00

APPENDIX B

Site Plan Showing Borehole Locations

Borehole Logs



LEGEND:

E. – EAST
ha – HECTARES
M. – MOUND OR MERIDIAN
N. – NORTH
RGE. – RANGE
R/W – RIGHT OF WAY
S. – SOUTH
SEC. – SECTION
TWP. – TOWNSHIP
W. – WEST
BOREHOLE WITH STAND PIPE PIEZOMETER . . . ○
BOREHOLES WITHOUT STAND PIPE PIEZOMETER . . . ⊗

ELK ISLAND PUBLIC SCHOOLS

BOREHOLE LOCATIONS PLAN

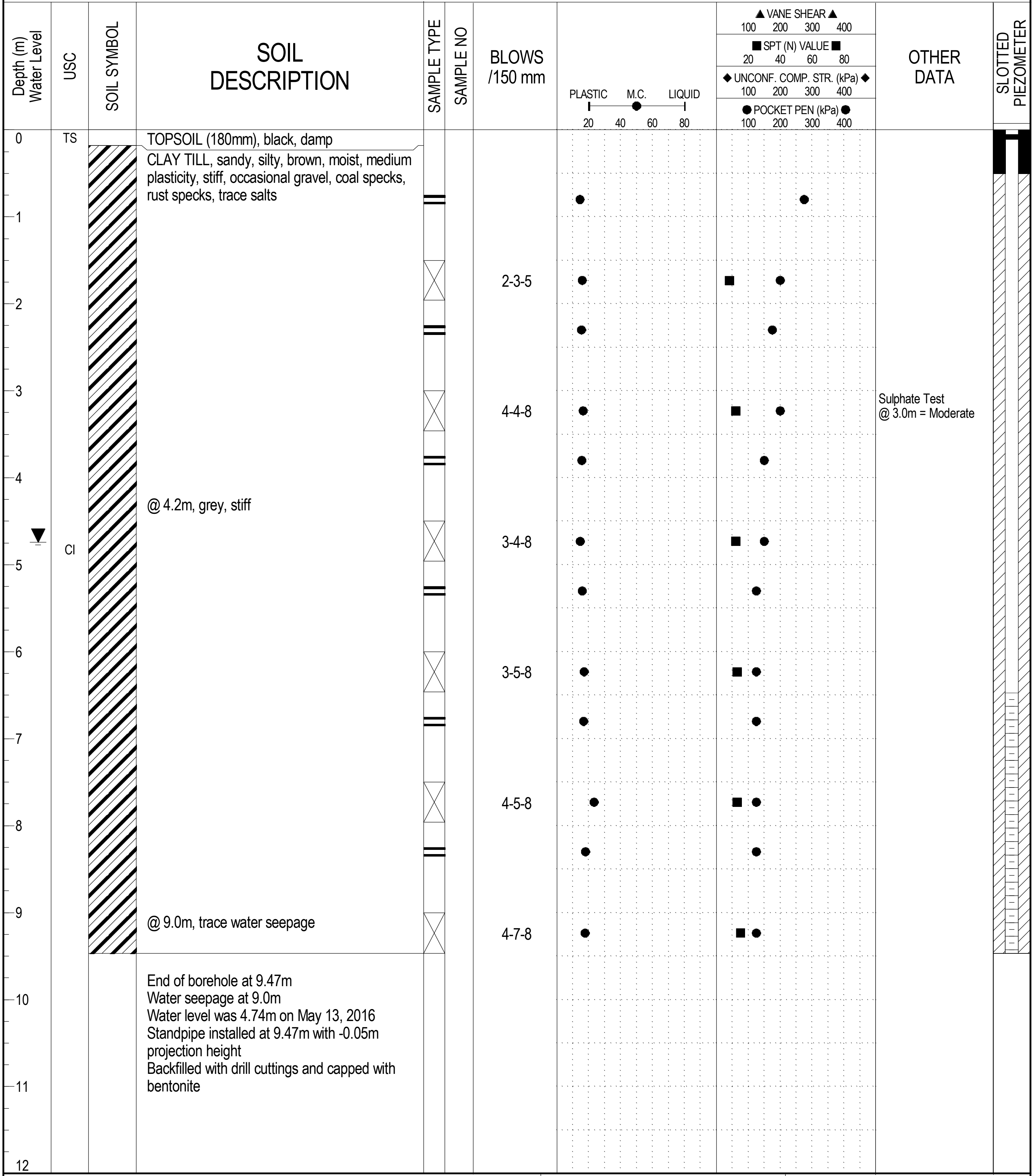
DAVIDSON CREEK NEW ELEMENTARY SCHOOL

OPUS

DRAWING NAME: 38765 GEO 01 REV 0		DATE: MAY 17, 2016
CLIENT: ELK ISLAND PUBLIC SCHOOLS		FILE No.: S–38765–00
DRAFTED BY: AM	SCALE: N.T.S.	CHECKED BY: AM

PROJECT: Proposed Davidson Creek K-6 School	LOCATION: Davidson Creek Park, Sherwood Park, AB	BOREHOLE NO: **BH16-01**
CLIENT: Elk Island Public Schools		NORTHING: 5936008
CONSULTANT PROJECT NO: S-38765-00	DRILL/METHOD: SOLID STEM AUGER	EASTING: 350248

SAMPLE TYPE	■ SHELBY TUBE	◪ CORE SAMPLE	☒ SPT SAMPLE	☰ GRAB SAMPLE	▥ NO RECOVERY	
BACKFILL TYPE	■ BENTONITE	⁘ PEA GRAVEL	▥ SLOUGH	⁘ GROUT	▨ DRILL CUTTINGS	⁘ SAND



TOPSOIL (180mm), black, damp

CLAY TILL, sandy, silty, brown, moist, medium plasticity, stiff, occasional gravel, coal specks, rust specks, trace salts

@ 4.2m, grey, stiff

@ 9.0m, trace water seepage

End of borehole at 9.47m
Water seepage at 9.0m
Water level was 4.74m on May 13, 2016
Standpipe installed at 9.47m with -0.05m projection height
Backfilled with drill cuttings and capped with bentonite

Sulphate Test
@ 3.0m = Moderate

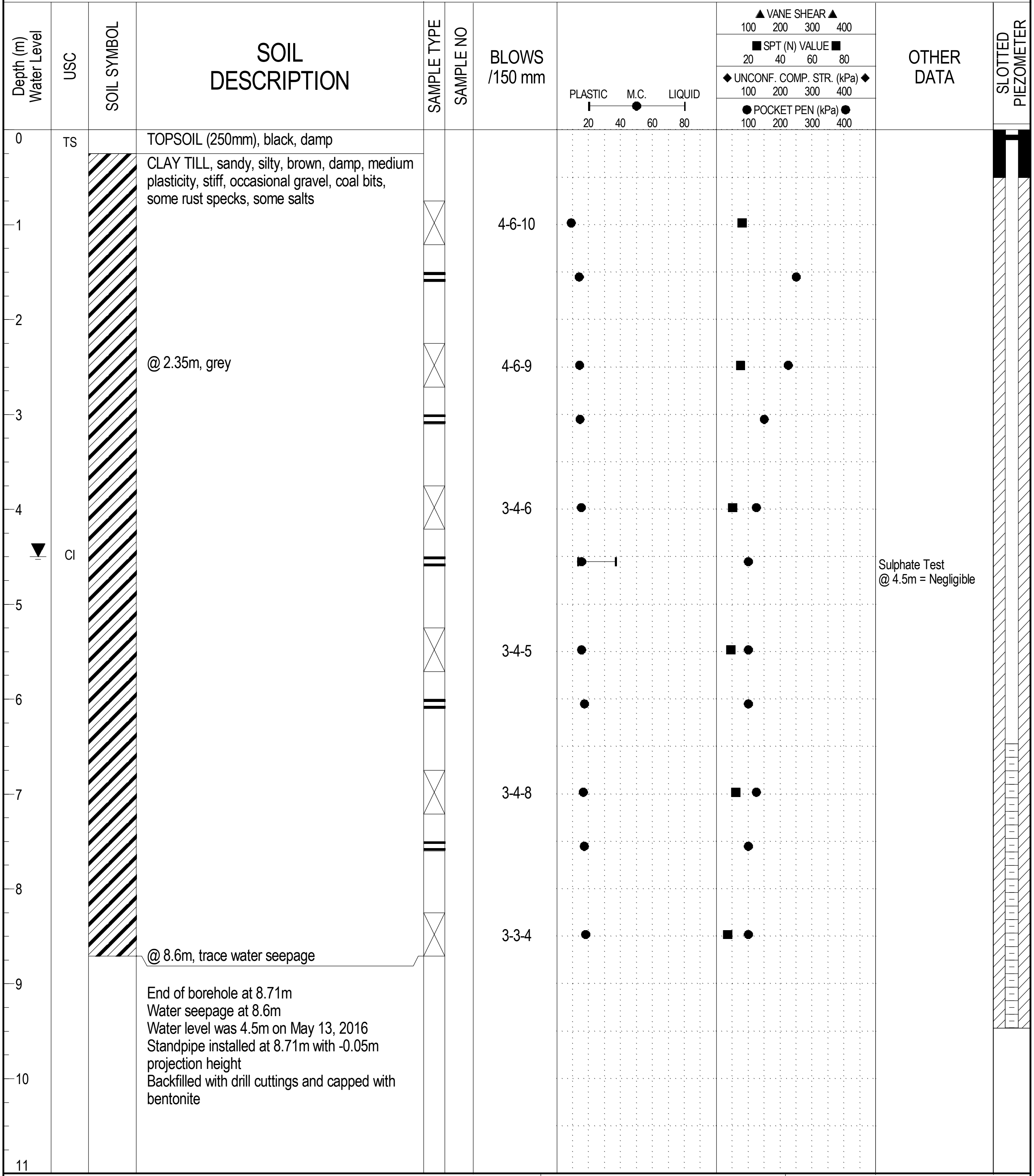
OPUS

LOGGED BY: DJ	COMPLETION DEPTH: 9.47 m
REVIEWED BY: NL	COMPLETION DATE: 5/2/16

PROJECT: Proposed Davidson Creek K-6 School	LOCATION: Davidson Creek Park, Sherwood Park, AB	BOREHOLE NO: **BH16-02**
CLIENT: Elk Island Public Schools		NORTHING: 5936061
CONSULTANT PROJECT NO: S-38765-00	DRILL/METHOD: SOLID STEM AUGER	EASTING: 350227

SAMPLE TYPE: ■ SHELBY TUBE ◩ CORE SAMPLE ☒ SPT SAMPLE ☰ GRAB SAMPLE ▥ NO RECOVERY

BACKFILL TYPE: ■ BENTONITE ⁘ PEA GRAVEL ▥ SLOUGH ⁘ GROUT ▨ DRILL CUTTINGS ⁘ SAND



TOPSOIL (250mm), black, damp

CLAY TILL, sandy, silty, brown, damp, medium plasticity, stiff, occasional gravel, coal bits, some rust specks, some salts

@ 2.35m, grey

@ 8.6m, trace water seepage

End of borehole at 8.71m
Water seepage at 8.6m
Water level was 4.5m on May 13, 2016
Standpipe installed at 8.71m with -0.05m projection height
Backfilled with drill cuttings and capped with bentonite

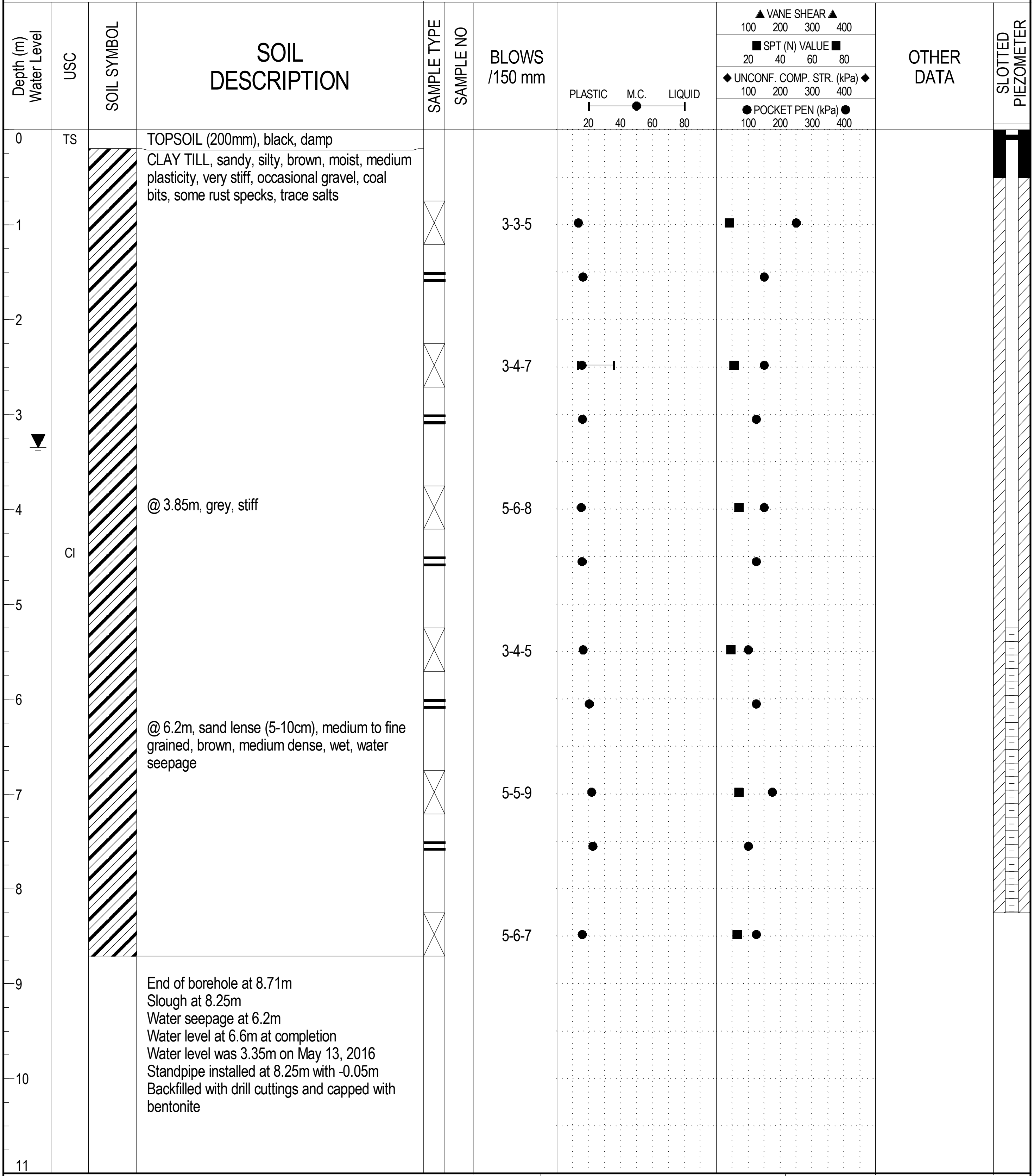
Sulphate Test
@ 4.5m = Negligible

LOGGED BY: DJ	COMPLETION DEPTH: 8.71 m
REVIEWED BY: NL	COMPLETION DATE: 5/2/16

PROJECT: Proposed Davidson Creek K-6 School	LOCATION: Davidson Creek Park, Sherwood Park, AB	BOREHOLE NO: **BH16-03**
CLIENT: Elk Island Public Schools		NORTHING: 5936020
CONSULTANT PROJECT NO: S-38765-00	DRILL/METHOD: SOLID STEM AUGER	EASTING: 350297

SAMPLE TYPE ■ SHELBY TUBE ◩ CORE SAMPLE ☒ SPT SAMPLE ☰ GRAB SAMPLE ▥ NO RECOVERY

BACKFILL TYPE ■ BENTONITE ⛶ PEA GRAVEL ▥ SLOUGH ⛶ GROUT ▨ DRILL CUTTINGS ⛶ SAND



Depth (m)	USC	Soil Description	Blows /150 mm
0	TS	TOPSOIL (200mm), black, damp	
	CI	CLAY TILL, sandy, silty, brown, moist, medium plasticity, very stiff, occasional gravel, coal bits, some rust specks, trace salts	
1			3-3-5
2.5			3-4-7
3.85		@ 3.85m, grey, stiff	5-6-8
5.5			3-4-5
6.2		@ 6.2m, sand lense (5-10cm), medium to fine grained, brown, medium dense, wet, water seepage	
7			5-5-9
8.5			5-6-7

End of borehole at 8.71m
Slough at 8.25m
Water seepage at 6.2m
Water level at 6.6m at completion
Water level was 3.35m on May 13, 2016
Standpipe installed at 8.25m with -0.05m
Backfilled with drill cuttings and capped with bentonite

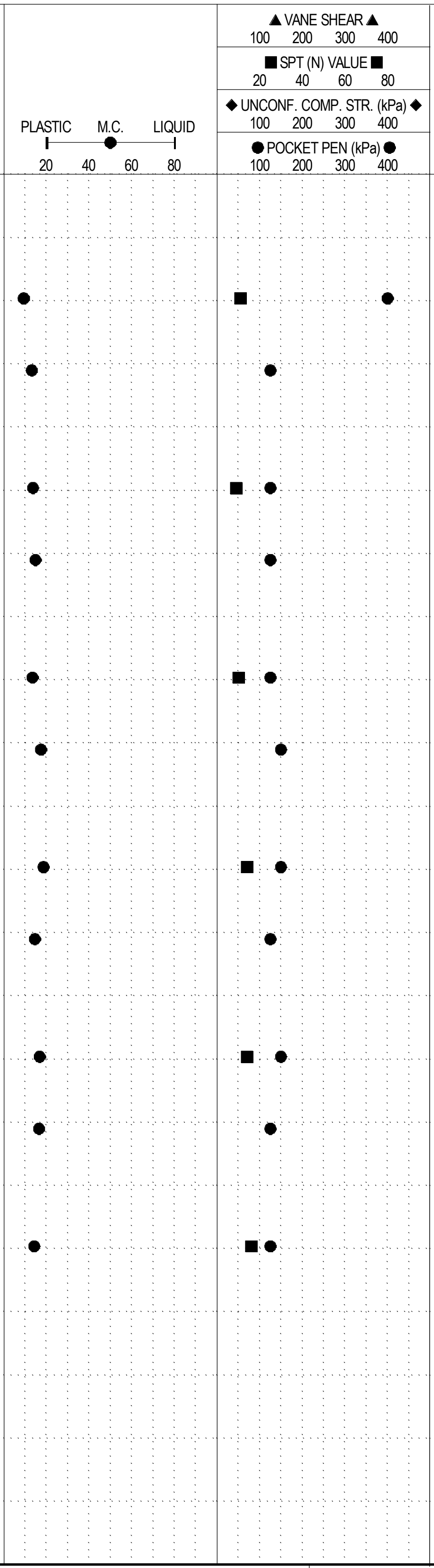
LOGGED BY: DJ	COMPLETION DEPTH: 8.71 m
REVIEWED BY: NL	COMPLETION DATE: 5/2/16

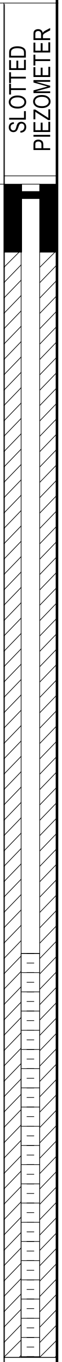
PROJECT: Proposed Davidson Creek K-6 School	LOCATION: Davidson Creek Park, Sherwood Park, AB	BOREHOLE NO: **BH16-04**
CLIENT: Elk Island Public Schools		NORTHING: 5935956
CONSULTANT PROJECT NO: S-38765-00	DRILL/METHOD: SOLID STEM AUGER	EASTING: 350285

SAMPLE TYPE ■ SHELBY TUBE ◪ CORE SAMPLE ☒ SPT SAMPLE ☰ GRAB SAMPLE ▥ NO RECOVERY

BACKFILL TYPE ■ BENTONITE ⁘ PEA GRAVEL ▥ SLOUGH ⁘ GROUT ▨ DRILL CUTTINGS ⁘ SAND

Depth (m) Water Level	USC	SOIL SYMBOL	SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NO	BLOWS /150 mm	OTHER DATA
0	TS		TOPSOIL (180mm), black, damp				
		▨	CLAY TILL, sandy, silty, brown, moist, medium plasticity, stiff, occasional gravel, coal bits, some rust, trace salts				
1				☒		4-5-6	
				☰			Sulphate Test @ 1.5m = Negligible
2				☒		2-4-5	
3 ▼				☰			
4				☒		3-4-6	
	CI		@ 4.3m, grey, stiff	☰			
5			@ 5.4m, sand lense (5-10cm), medium to fine grained, brown, medium dense, wet, water seepage	☒		5-7-7	
6				☰			
7				☒		5-6-8	
				☰			
8				☒		4-6-10	
9			End of borehole at 8.71m Slough at 8.25m Water seepage was at 5.4m Water level was 7.3m at completion Water level was 3.25m on May 13, 2016 Standpipe installed at 8.71m with -0.05m projection height Backfilled with drill cuttings and capped with bentonite				
10							
11							



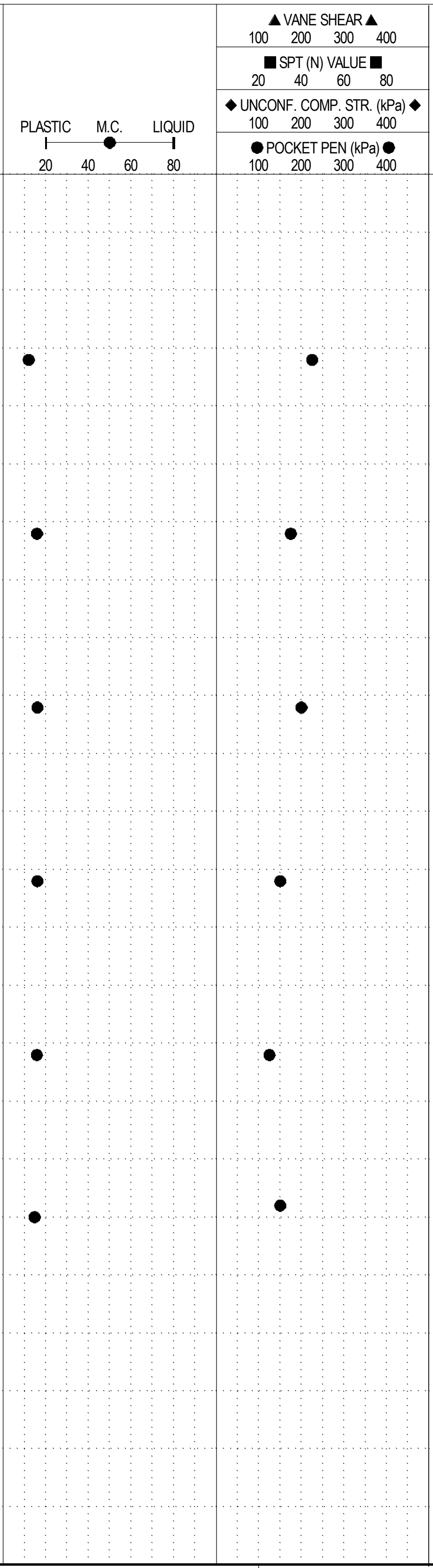


LOGGED BY: DJ	COMPLETION DEPTH: 8.71 m
REVIEWED BY: NL	COMPLETION DATE: 5/2/16

PROJECT: Proposed Davidson Creek K-6 School	LOCATION: Davidson Creek Park, Sherwood Park, AB	BOREHOLE NO: **BH16-05**
CLIENT: Elk Island Public Schools		NORTHING: 5936059
CONSULTANT PROJECT NO: S-38765-00	DRILL/METHOD: SOLID STEM AUGER	EASTING: 350263

SAMPLE TYPE ■ SHELBY TUBE ◩ CORE SAMPLE ☒ SPT SAMPLE ☰ GRAB SAMPLE ▥ NO RECOVERY

Depth (m)	USC	SOIL SYMBOL	SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NO	BLOWS /150 mm	OTHER DATA
0	TS		TOPSOIL (200mm), black, damp				
	CI	▨	CLAY TILL, sandy, silty, brown, moist, medium plasticity, stiff, occasional gravel, coal bits, some rust specks, trace salts	☰ ☰ ☰ ☰ ☰ ☰			
			End of borehole at 4.5m Borehole dry at completion Standpipe not installed Backfilled with drill cuttings				

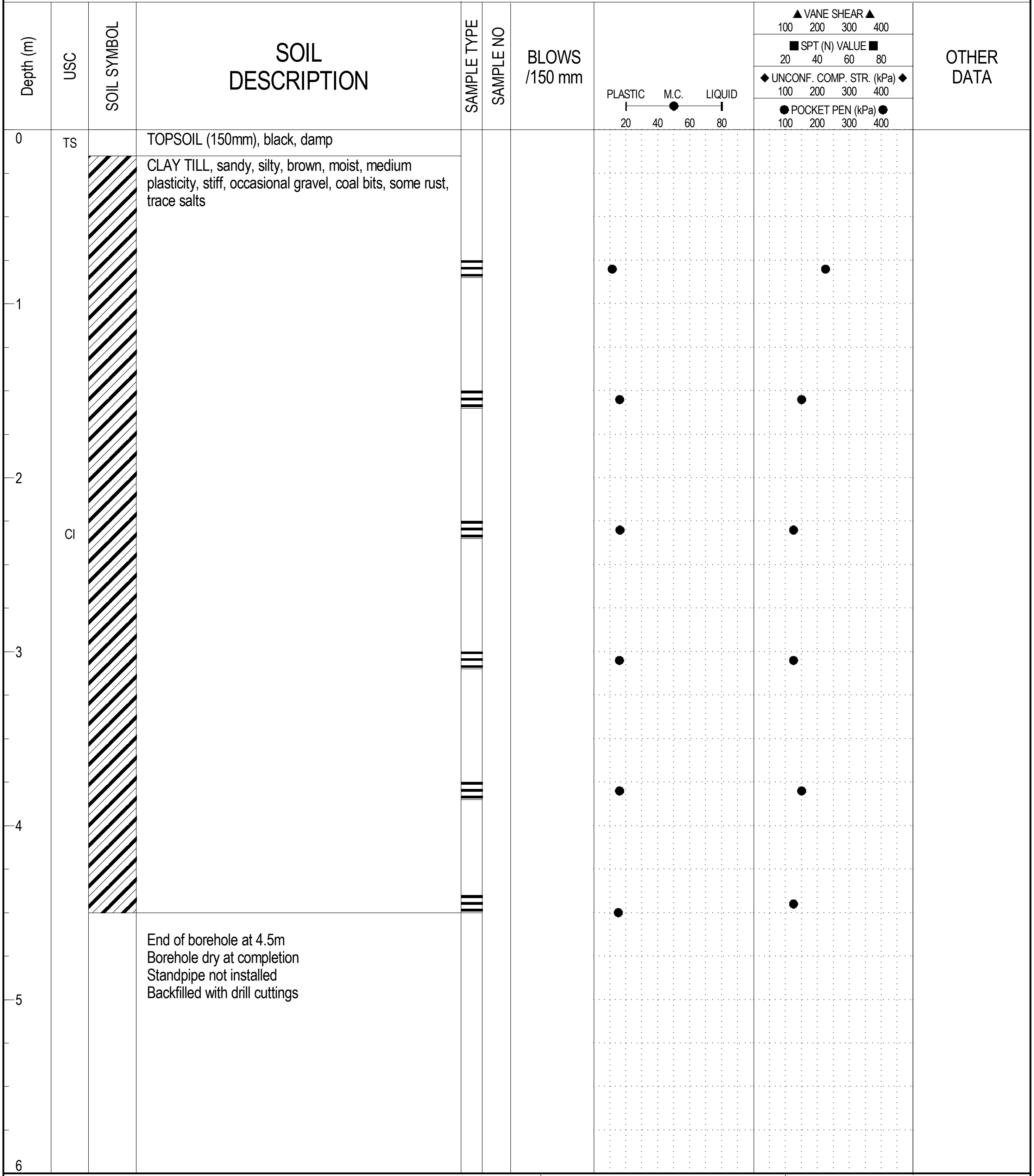


Depth (m): 0, 1, 2, 3, 4, 5, 6

LOGGED BY: DJ	COMPLETION DEPTH: 4.50 m
REVIEWED BY: NL	COMPLETION DATE: 5/2/16

PROJECT: Proposed Davidson Creek K-6 School
LOCATION: Davidson Creek Park, Sherwood Park, AB
BOREHOLE NO: **BH16-06**
CLIENT: Elk Island Public Schools
NORTHING: 5935985
CONSULTANT PROJECT NO: S-38765-00
DRILL/METHOD: SOLID STEM AUGER
EASTING: 350306

SAMPLE TYPE ■ SHELBY TUBE ◪ CORE SAMPLE ☒ SPT SAMPLE ☰ GRAB SAMPLE ▥ NO RECOVERY



TOPSOIL (150mm), black, damp

CLAY TILL, sandy, silty, brown, moist, medium plasticity, stiff, occasional gravel, coal bits, some rust, trace salts

End of borehole at 4.5m
Borehole dry at completion
Standpipe not installed
Backfilled with drill cuttings

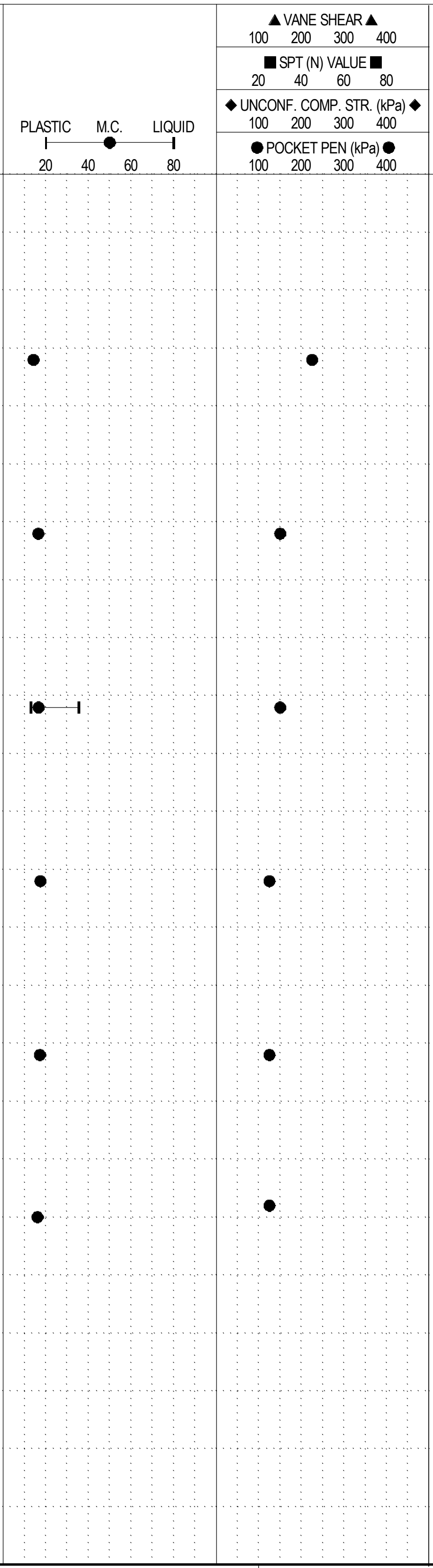
OPUS

LOGGED BY: DJ
REVIEWED BY: NL
COMPLETION DEPTH: 4.50 m
COMPLETION DATE: 5/2/16

PROJECT: Proposed Davidson Creek K-6 School	LOCATION: Davidson Creek Park, Sherwood Park, AB	BOREHOLE NO: **BH16-07**
CLIENT: Elk Island Public Schools		NORTHING: 5935961
CONSULTANT PROJECT NO: S-38765-00	DRILL/METHOD: SOLID STEM AUGER	EASTING: 350242

SAMPLE TYPE ■ SHELBY TUBE ◩ CORE SAMPLE ☒ SPT SAMPLE ☰ GRAB SAMPLE ▥ NO RECOVERY

Depth (m)	USC	SOIL SYMBOL	SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NO	BLOWS /150 mm	OTHER DATA
0	TS		TOPSOIL (160mm), black, damp				
	CI	▨	CLAY TILL, sandy, silty, brown, moist, medium plasticity, stiff, occasional gravel, coal bits, some rust specks, trace salts	☰ ☰ ☰ ☰ ☰ ☰			
4.5			End of borehole at 4.5m Borehole dry at completion Standpipe not installed Backfilled with drill cuttings				

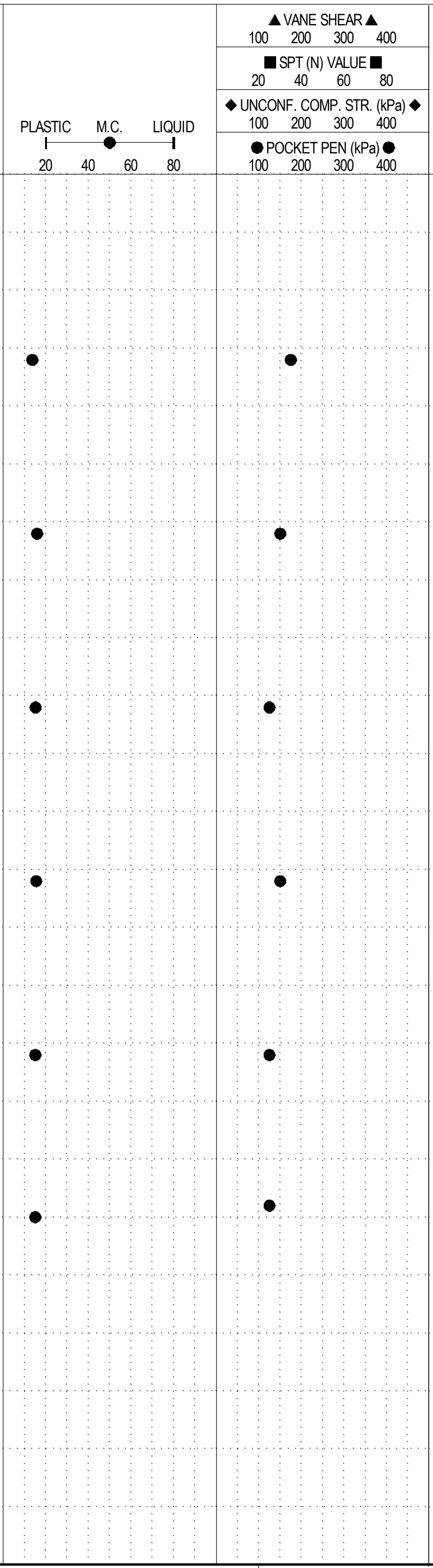


LOGGED BY: DJ	COMPLETION DEPTH: 4.50 m
REVIEWED BY: NL	COMPLETION DATE: 5/2/16

PROJECT: Proposed Davidson Creek K-6 School	LOCATION: Davidson Creek Park, Sherwood Park, AB	BOREHOLE NO: **BH16-08**
CLIENT: Elk Island Public Schools		NORTHING: 5936023
CONSULTANT PROJECT NO: S-38765-00	DRILL/METHOD: SOLID STEM AUGER	EASTING: 350195

SAMPLE TYPE ■ SHELBY TUBE ◩ CORE SAMPLE ☒ SPT SAMPLE ☰ GRAB SAMPLE ▥ NO RECOVERY

Depth (m)	USC	SOIL SYMBOL	SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NO	BLOWS /150 mm	OTHER DATA
0	TS		TOPSOIL (200mm), black, damp				
	CI	▨	CLAY TILL, sandy, silty, brown, moist, medium plasticity, stiff, occasional gravel, coal bits, some rust, trace salts @ 3.1m, grey, stiff	☰ ☰ ☰ ☰ ☰ ☰			
			End of borehole at 4.5m Borehole dry at completion Standpipe not installed Backfilled with drill cuttings				

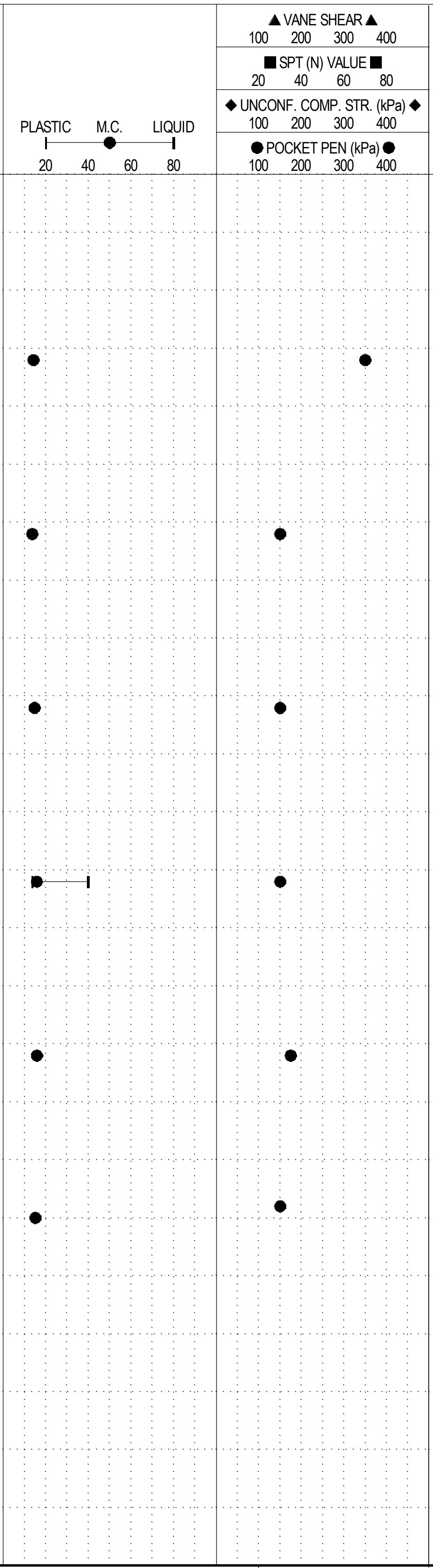


LOGGED BY: DJ	COMPLETION DEPTH: 4.50 m
REVIEWED BY: NL	COMPLETION DATE: 5/2/16

PROJECT: Proposed Davidson Creek K-6 School	LOCATION: Davidson Creek Park, Sherwood Park, AB	BOREHOLE NO: **BH16-09**
CLIENT: Elk Island Public Schools		NORTHING: 5936065
CONSULTANT PROJECT NO: S-38765-00	DRILL/METHOD: SOLID STEM AUGER	EASTING: 350190

SAMPLE TYPE ■ SHELBY TUBE ◪ CORE SAMPLE ☒ SPT SAMPLE ☰ GRAB SAMPLE ▥ NO RECOVERY

Depth (m)	USC	SOIL SYMBOL	SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NO	BLOWS /150 mm	OTHER DATA
0	TS		TOPSOIL (200mm), black, damp				
	CI	▨	CLAY TILL, sandy, silty, brown, moist, medium plasticity, stiff, occasional gravel, coal bits, some rust, trace salts				
0.8				☰			
1.5				☰			
2			@ 2.0m, sand lense (5-10cm), medium to fine grained, brown, damp, loose				
2.3				☰			
			@ 2.7m, grey, stiff				
3				☰			
3.8				☰			
4.5			End of borehole at 4.5m Borehole dry at completion Standpipe not installed Backfilled with drill cuttings	☰			



LOGGED BY: DJ	COMPLETION DEPTH: 4.50 m
REVIEWED BY: NL	COMPLETION DATE: 5/2/16

APPENDIX C

Laboratory Test Results

PROJECT: Proposed Davidson Creek K-6 School | CLIENT: Elk Island Public Schools | PROJECT NO: S-38765-00

Borehole No.	Depth (m)	Atterberg Limits Liquid Limit	Plastic Limit	Plasticity Index	Field Moisture Content (%)	Liquidity Index	Estimated Optimum Moisture (%)	Estimated Maximum Density (kg/m^3)	Soil Class	Potential Frost Action	Potential Erosion Resistance
16-01	0.75				14.6						
16-01	1.50				16.0						
16-01	2.25				15.5						
16-01	3.00				16.5						
16-01	3.75				15.6						
16-01	4.50				14.7						
16-01	5.25				15.9						
16-01	6.00				17.1						
16-01	6.75				16.8						
16-01	7.50				23.4						
16-01	8.25				18.0						
16-01	9.00				17.7						
16-02	0.75				9.0						
16-02	1.50				14.0						
16-02	2.25				14.2						
16-02	3.00				14.5						
16-02	3.75				15.3						
16-02	4.50	37	13	24	15.6	0.10	14	1880	CI	M-H	F-G
16-02	5.25				15.5						
16-02	6.00				17.3						
16-02	6.75				16.6						
16-02	7.50				17.2						
16-02	8.25				18.1						
16-03	0.75				13.6						
16-03	1.50				16.3						
16-03	2.25	36	13	23	15.7	0.11	13	1890	CI	M-H	F-G
16-03	3.00				16.1						
16-03	3.75				15.3						
16-03	4.50				15.8						
16-03	5.25				16.5						
16-03	6.00				20.4						
16-03	6.75				21.9						
16-03	7.50				22.6						
16-03	8.25				15.9						
16-04	0.75				9.4						
16-04	1.50				13.1						
16-04	2.25				13.7						
16-04	3.00				14.9						
16-04	3.75				13.5						
16-04	4.50				17.4						
16-04	5.25				18.6						
16-04	6.00				14.6						
16-04	6.75				16.9						

Potential Frost Action : None (N), Very Slight (VS), Slight (S), Medium (M), High (H), Very High (VH)
Potential Erosion Resistance : Excellent (E), Good (G), Fair (F), Poor (P)

Sheet 1 of 2

SUMMARY OF LABORATORY RESULTS

PROJECT: Proposed Davidson Creek K-6 School CLIENT: Elk Island Public Schools PROJECT NO: S-38765-00

Borehole No.	Depth (m)	Atterberg Limits			Field Moisture Content (%)	Liquidity Index	Estimated Optimum Moisture (%)	Estimated Maximum Density (kg/m^3)	Soil Class	Potential Frost Action	Potential Erosion Resistance
		Liquid Limit	Plastic Limit	Plasticity Index							
16-04	7.50				16.5						
16-04	8.25				14.3						
16-05	0.75				12.0						
16-05	1.50				15.8						
16-05	2.25				16.0						
16-05	3.00				16.0						
16-05	3.75				15.8						
16-05	4.50				14.7						
16-06	0.75				11.4						
16-06	1.50				16.0						
16-06	2.25				16.2						
16-06	3.00				15.9						
16-06	3.75				16.0						
16-06	4.50				15.2						
16-07	0.75				14.2						
16-07	1.50				16.5						
16-07	2.25	35	13	22	16.6	0.17	13	1910	CI	M-H	F-G
16-07	3.00				17.4						
16-07	3.75				17.2						
16-07	4.50				16.1						
16-08	0.75				13.6						
16-08	1.50				15.9						
16-08	2.25				15.2						
16-08	3.00				15.5						
16-08	3.75				15.1						
16-08	4.50				15.0						
16-09	0.75				14.2						
16-09	1.50				13.7						
16-09	2.25				14.7						
16-09	3.00	40	14	26	15.8	0.09	14	1870	CI	M-H	F-G
16-09	3.75				15.8						
16-09	4.50				15.1						

Potential Frost Action : None (N), Very Slight (VS), Slight (S), Medium (M), High (H), Very High (VH)
Potential Erosion Resistance : Excellent (E), Good (G), Fair (F), Poor (P)

SUMMARY OF LABORATORY RESULTS