



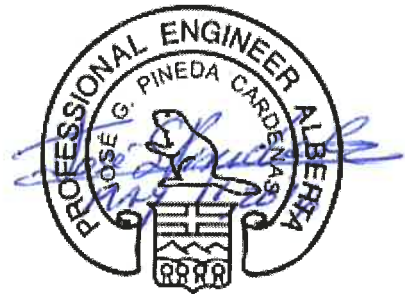
THURBER ENGINEERING LTD.

**ROSSDALE CENTRE PHASE 1
EDMONTON, ALBERTA
GEOTECHNICAL INVESTIGATION**

Report

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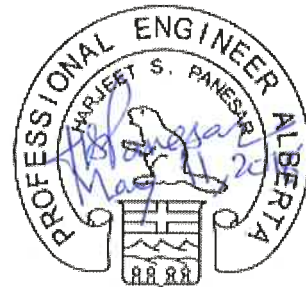
DUB ARCHITECTS LTD.



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The Association of Professional Engineers, Geologists and Geophysicists of Alberta

Date: May 11, 2016
File: 12553



Harjeet Panesar, M.Sc., P. Eng.
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STATEMENT OF LIMITATIONS AND CONDITIONS

APPENDIX A

- Drawing 12553-1 - Site Plan Showing Approximate Test Hole Locations

APPENDIX B

- Symbols and Terms Used on Test Hole Logs
- Modified Unified Soils Classification
- Test Hole Logs

APPENDIX C

- Recommended Construction Procedures



1. INTRODUCTION

This report presents the results of a geotechnical investigation undertaken by Thurber Engineering Ltd. (Thurber) for the proposed Rossdale Centre Phase 1 building located at 9856 - 100 Street NW, Edmonton, Alberta.

The geotechnical investigation was carried out in general accordance with our proposal letter dated March 28, 2016 to Mr. Norman Kloeber of DUB Architects Limited (DUB). Authorization to proceed with the work was received from Mr. Kloeber via an e-mail dated March 31, 2016.

The scope of work did not include any environmental soil and groundwater contamination assessment.

It is a condition of this report that Thurber's performance of its professional services is subject to the attached Statement of Limitations and Conditions.

2. PROJECT DESCRIPTION

Based on the information provided to Thurber, we understand that the project comprises construction of a new wood frame five storey residential building. The building will also have a 1.5 level underground concrete parking structure.

3. GEOTECHNICAL INVESTIGATION

3.1 Field Drilling Program

Five test holes TH16-01 through -05 were drilled on April 20, 2016 at the project site to depths ranging between 6.9 and 10.4 m below existing ground surface, using a truck mounted drill rig operated by All Service Drilling Inc. Approximate test hole locations are shown on Drawing 12553-1, in Appendix A.

Prior to commencing the drilling program, Alberta-One-Call was contacted and the test hole locations were cleared of underground utilities.

Disturbed soil samples were obtained from the auger flights during drilling and Standard Penetration Tests (SPTs) were carried out at selected depths in the test holes. The undrained shear strength (C_{pen} value) of cohesive samples was estimated using a pocket penetrometer. Water levels were noted during and after completion of the drilling.



On completion of drilling, three standpipe piezometers were installed within the project site to allow future groundwater levels monitoring. The standpipe piezometers were monitored upon completion of the installation, and again on April 25, 2016.

3.2 Laboratory Testing

Laboratory testing included visual classification and determination of the natural moisture content of all soil samples. In addition, Atterberg Limits and water soluble sulphate content tests were carried out on selected representative soil samples.

The results of the drilling and laboratory program are summarized on the test holes logs in Appendix B. An explanation of the symbols and terms used to describe observations in the test holes logs and the Modified Unified Soil Classification are also provided in Appendix B.

4. SITE DESCRIPTION

4.1 Surface Conditions

The site is generally flat. At the time of undertaking the field drilling program, the site was covered with grass vegetation.

4.2 Subsurface Conditions

The general subsurface conditions encountered at the test hole locations, in descending order, consisted of the following main stratigraphy:

- Topsoil and Organics
- Clay
- Sand
- Gravel
- Bedrock.

Further descriptions of the main soil layers is provided in the following sections. A detailed description of subsurface conditions observed at each test hole location is presented on the test hole logs in Appendix B. The detailed description in Appendix B should be used in preference to the generalized descriptions given below.



4.2.1 Topsoil and Organics

Topsoil and organics consisting of black, silty clay with trace to some rootlets was encountered starting from surface and extended to depths varying between 0.15 m and 0.6 m below ground surface.

4.2.2 Clay

Clay was encountered below the topsoil and extended to depths approximately ranging between 3.3 m and 6.4 m. The clay was generally brown to grey, silty, and contained some sand. The natural moisture content of the clay ranged from about 12 to 28 percent. SPT N values in the clay ranged from 4 to 9 blows per 300 mm penetration, indicating a soft to firm consistency.

One Atterberg Limits test conducted on the clay sample determined a liquid and plastic limit of 48 and 15 percent respectively. Visual observations and the Atterberg Limits test results indicate that the clay is of high plasticity near the surface and becomes low plastic with depth.

It should be noted that the clay in the upper 4 m is prone to swelling or shrinkage in response to water content changes.

4.2.3 Sand

Sand was encountered below the clay and extended to a depth of approximately 6.8 m below ground surface. The sand was generally grey, silty, and contained traces of clay. The natural moisture content of the sand ranged from about 7 to 23 percent. SPT N values in the sand ranged from 4 to 8 blows per 300 mm penetration, indicating a loose relative density.

4.2.4 Gravel

Gravel was encountered below the sand and extended to a depth of approximately 9.5 m below ground surface. The gravel was brown and grey, silty, and contained traces of clay and cobbles. The natural moisture content of the gravel ranged from about 7 to 20 percent. SPT N values in the gravel ranged from 28 to 54 blows per 300 mm penetration, indicating a compact to very dense relative density.

4.2.5 Bedrock

Bedrock consisting of clay shale and sandstone was encountered below the gravel in test holes TH16-01 and -03 at a depth of about 9.5 m. The natural moisture content of the bedrock ranged



from approximately 20 to 22 percent. SPT N values in the bedrock ranged from 35 to 52 blows per 300 mm penetration, indicating a hard to very hard consistency.

4.3 Groundwater Conditions

Depths of groundwater seepage encountered in the test holes during drilling are shown on the test hole logs in Appendix B. Table 4.1 summarizes the groundwater levels measured during the field program and again on April 25, 2016.

**TABLE 4.1
ROSSDALE PHASE I BUILDING
SUMMARY OF GROUNDWATER, SEEPAGE, AND SLOUGH CONDITIONS**

TEST HOLE	TEST HOLE DEPTH (m) B.G.S.*	PIEZOMETER DEPTH (m) B.G.S.*	SEEPAGE (m) B.G.S.*	SLOUGH ON COMPLETION (m) B.G.S.*	FREE WATER ON COMPLETION B.G.S* (m)	GROUNDWATER LEVEL IN STANDPIPE PIEZOMETERS April 25, 2016 (m) B.G.S.*
TH16-01	10.4	7.5	5.5	7.5	Dry	6.8
TH16-02	6.9	N/A	5.3	6.4	6.2	N/A
TH16-03	10.4	7.2	6.9	7.2	7.0	7.2
TH16-04	6.9	N/A	6.9	None	6.9	N/A
TH16-05	10.4	7.8	6.3	8.1	7.0	6.9

* Below existing ground surface

Based on the short term groundwater level readings in the standpipe piezometers, the groundwater levels at the site range from 6.8 m to 7.2 m below ground surface.

It should be noted that the groundwater levels can vary in response to seasonal factors and precipitation; hence, the actual groundwater conditions at the time of construction could vary from those recorded during this investigation.

4.4 Frost Effects

The clay is expected to have low frost susceptibility, but still may be subjected to some heaving during winter, and softening and strength loss as a result of thaw. Since the clay was continuous from ground surface the average annual depth of frost penetration is estimated to be about 1.6 m, based on an average air freezing index of 1400 degree-days Celsius. Based on an air freezing index of 2200 degree-days Celsius for a 50 year return period, the maximum depth of frost penetration is estimated to be about 2.2 m.



The estimated depths of frost penetration are based on a uniform soil type with no snow/insulation cover. If the area is covered with turf or significant snow cover, the depth of frost penetration will be less. The mean annual freezing index could be used for construction with some risk; the 50 year return index is usually chosen for long term design of foundations.

5. GEOTECHNICAL RECOMMENDATIONS

5.1 Site Evaluation

Results of the geotechnical investigation indicate that the proposed Rossdale Phase I building site is underlain by a sequence of clay, sand, gravel and bedrock in descending order.

Available information indicates that the proposed building will be founded on firm clay at a depth of about 3 m below ground surface. We understand that the preferred foundation for the proposed building are spread footings. However, we have also provided recommendations for cast-in-place concrete piles.

We also understand that cut and fill in the order of about 1 m around the proposed building will be required as part of the proposed development. The new building will be located at distance of about 10 m from Rossdale Road and 101 Street.

Recommendations for site grading, temporary excavation, drainage, backfilling and foundation design parameters are provided in the following sections.

5.2 Site Grading

We understand that about 1 m of cut and fill outside the building perimeter may be required as part of the site grading. All topsoil, organics and deleterious material should be removed from below the new fill areas. Qualified geotechnical personnel should inspect the base of the excavation to confirm that all unsuitable materials have been removed.

Engineered fill (if required) for site grading should consist of low to medium plastic clay placed and compacted in lifts not exceeding 150 mm in compacted thickness. The uniformity and compactive effort of the engineered fill are important in minimizing the potential for differential settlement. The following recommendations are provided for fill placement and compaction:

- All site grading fill to be placed under any access road or sidewalk should be placed in maximum 150 mm thick lifts and compacted to at least 98 percent of Standard Proctor



Maximum Dry Density (SPMDD) within +/-2 percent of Optimum Moisture Content (OMC).

- All fill used for landscaping purposes needs only moderate compaction (i.e., 92 percent of SPMDD) such that future settlements do not affect site drainage.

Frozen soil should not be used for backfill. Clean (<5 percent fines) well-graded, gravel should be used as backfill in cold weather conditions.

The lift thickness and in-situ density of compacted fills should be confirmed by field density test measurements during construction.

It is recommended that the finished subgrade below pavements and sidewalks be sloped at a minimum gradient of 1 percent toward catch basins or ditches to drain subsurface water away from the roadways and structures. This will reduce the likelihood of ponding of water which could result in frost heaving of the clay subgrade.

5.3 Excavation Considerations

5.3.1 General

We understand that the building will have an underground parkade which will be about 3 m below ground surface.

Open excavations are considered feasible to construct the underground parkade. Temporary shoring may be used to support the excavation walls where sufficient space is not available.

5.3.2 Excavation

Temporary excavation slopes through the clay for the underground parkade should be sloped not steeper than 1H:1V. Flatter slopes will likely be required to maintain the stability of open excavations if soft clay or water bearing silt, gravel and sand layers are encountered during excavations.

Excavated material should be kept back from the top of the excavation by at least a distance equal to the excavation depth.

Groundwater seepage was not noticed in the upper 4 m. Therefore, if minor seepage is encountered within the expected excavation depth of 3 m, the groundwater flow is expected to be of a magnitude that could be typically handled by a sump and pump.



It is recommended that the excavation be carried out in a timely manner and backfilled as soon as possible. The ground surface outside the excavation should be sloped at a grade of at least 2 percent to shed water away from the building. Consideration should be given to covering the excavated slopes with tarps to reduce erosion, raveling, and sloughing of excavation walls during periods of rain.

5.3.3 Backfill

Free-draining clean granular material (less than 5 percent passing a No. 200 sieve) would be the first choice of material for backfilling against the underground parkade walls as it compacts relatively easily and does not settle significantly with time. The backfill should be carefully placed against the parkade walls to avoid over-compaction of the material and distress to the walls, and should be capped with an impervious barrier, such as a compacted clay layer of about 300 mm thickness. A recommended typical gradation for free draining granular material is provided in Table 5.4.

The on-site clay material is not as desirable for backfilling because it remains in hard chunks and it is difficult to obtain uniform compaction resulting in non-uniform ground settlement with time. If, however, it is necessary to use clay material against the basement walls, then it should be free from organic content and should be broken down with no large chunks of clay remaining. The clay should be carefully placed and hand tamped in lifts of 150 mm or less to ensure uniform compaction.

Backfilling should be delayed until the concrete in the foundation wall has gained sufficient strength. Premature backfilling or backfilling with excessive force may crack the foundation walls. Heavy compaction equipment should not be allowed to operate within a distance of about 1.5 m from the face of the wall to prevent potential for overstressing of the basement walls.

The ground surface should be sloped at a grade of at least 2 percent away from the building to shed water away from the building.

5.3.4 Lateral Earth Pressures

A triangular earth pressure distribution may be used for design of underground structures resisting earth pressures. The parameters to be used will depend upon the extent and direction of movement of the soil, the nature and extent of the backfill, and groundwater conditions. The horizontal earth pressure, p_h , may be calculated as follows using the earth pressure coefficients given in Table 5.1 provided the backfilling is carried out as recommended in Section 5.3.3.



$$p_h = k [(W \times H) + q] \text{ (kPa)}$$

Where:

- k = the appropriate coefficient of earth pressure from Table 5.1
- W = the bulk unit weight, in kN/m³
- H = the depth below backfill surface, in m
- q = surcharge pressure (kPa)

TABLE 5.1
EARTH PRESSURE COEFFICIENTS FOR PERMANENT RETAINING WALL
ASSUMING VERTICAL WALL, GOOD SURFACE DRAINAGE, AND HORIZONTAL
BACKFILL

SOIL DESCRIPTION	BULK UNIT WEIGHT kN/m ³	ASSUMED FRICTION ANGLE	K _a ACTIVE	K _o AT-REST	K _p PASSIVE
Native Clay	20	23	0.44	0.61	2.30
Gravel Backfill (compacted to 98 percent of SPMDD)	21.5	33	0.30	0.46	3.40

For rigid foundation walls that are restrained from movement, the at-rest earth pressure should be used. Active earth pressures may be used for the design of relatively low retaining walls, which can be allowed to move laterally at the top a distance of 0.01 and 0.001 times the height of the wall, for cohesive and cohesionless material, respectively. Appropriate load factors should be applied to earth pressures in Limit States Design. The passive pressure will be mobilized when the top of the wall has moved into the backfill a distance of 0.02 times the height of the wall. A geotechnical resistance factor of 0.5 should be applied to the passive pressure determined from the values given in Table 5.1.

The earth pressures are governed by the soil type and the extent of selected backfill material behind the wall. To mobilize the recommended active earth pressure values, the mobilized soil zone behind the wall is typically defined by a wedge-shaped zone delineated by projecting a 1H:1V line to ground surface from a point located 0.5 m into the soil from the base of the wall footing. To mobilize the recommended passive earth pressure values, the mobilized soil zone behind the wall is typically defined by a wedge-shaped zone delineated by projecting a 2H:1V line from the base of the wall footing.



Where traffic or other live loads may travel or operate near the retaining wall the horizontal pressures due to the live load should be superimposed on the static earth pressures.

If foundations of buildings, or services exist behind the top of the wall at a distance smaller than the height of the wall and not closer than half the height, the pressure may be computed using a coefficient $K = 0.5(K_a + K_o)$. If foundations of buildings or services exists at shallow depth at a distance smaller than half the height behind the top of the wall, pressure may be computed using the coefficient of earth pressure at rest, K_o .

5.4 Building Foundations

5.4.1 General

The following foundation types are considered feasible for buildings on this site:

- Spread Footings
- Cast-in-place concrete friction and end bearing piles
- Continuous Flight Augers (CFA) piles

It should be noted that spread footings are generally more prone to long term movement, due to seasonal effects than pile foundations. Due to the presence of sand and gravel below clay, temporary casing will be required for the installation of cast-in-place concrete piles.

Recommendations for design of the above stated foundation types are provided in the following sections. Additional recommended construction procedures are provided in Appendix C.

5.4.2 Spread Footings

Spread footings should be designed and constructed in accordance to the following recommendations:

- All footings should be founded on the undisturbed firm clay or sand stratum at the expected footing depths of approximately 3 m below the existing grade.
- The clay at foundation level is sandy and silty and may be prone to disturbance under traffic. Any disturbed soils encountered in the footing excavations should be recompacted or removed and be replaced with lean concrete. Disturbed soil should not be allowed to remain in the footing excavations. A mud slab may be used at the base of

footing excavations to avoid base disturbance. Alternatively, the depth of footing excavation could be left about 0.5 m higher to prevent deterioration of the footing base and the material would be removed prior to construction of the footings.

- Spread footings may be designed using the ultimate and factored ULS bearing capacities provided below. The factored ULS bearing capacities are based on a geotechnical resistance factor of 0.5, in accordance with NBCC 2005.

**TABLE 5.2
ULTIMATE AND FACTORED ULS BEARING CAPACITIES
FOR SPREAD FOOTINGS**

FOOTING TYPE	ULTIMATE BEARING RESISTANCE (kPa)	FACTORED ULS BEARING RESISTANCE (kPa)
Strip	160	80
Square	190	95

- For preliminary design, an estimated modulus of subgrade reaction of 6 MPa/m may be used for spread footing supported on the firm clay or loose sand. The subgrade modulus should be modified for the actual widths used in the structural analysis. This should be verified by settlement analysis once more details on the foundation layout and loads are available.
- Where spread footings are constructed adjacent to temporary cut slopes, (i.e. adjacent to deeper elevator foundations) the slopes should be backfilled as soon as practical with fillcrete or compacted crushed gravel in order to maintain the required bearing resistance under the upper footings.
- Where necessary, temporary shoring of cut slopes or protection with plastic tarps may be used to protect the slopes from weathering and degradation.
- The excavated base of the foundation level should be protected from weathering and frost action to prevent the deterioration of the soil at footing level. Settlements could be greater if the soils are disturbed by construction activities or not properly protected.
- The footing excavations should be inspected prior to pouring of the concrete.

The ultimate friction factor between the concrete base of the foundation and sand/clay may be taken as 0.4.



5.4.3 Cast-in-Place Concrete Pile

Cast-in-place concrete piles may be designed and installed according to the following recommendations:

- a) End bearing pile bases should be founded at least 2 m into hard clay shale bedrock (12 m below the existing ground surface or 9 m below the underground parkade).
- b) Drilled cast-in-place reinforced concrete piles may be designed based on the factored ULS skin friction and end-bearing values provided in Table 5.3.

**TABLE 5.3
RECOMMENDED END BEARING AND SKIN FRICTION VALUES
FOR DRILLED CONCRETE END BEARING PILES**

AVERAGE DEPTH BELOW PARKADE* (m)	VERTICAL STATIC LOADING			
	Skin Friction (kPa)		End Bearing (kPa)	
	Ultimate	ULS Factored (0.4)	Ultimate	ULS Factored (0.4)
0 – 2	0	0	N/A	N/A
2 – 4	15	6	N/A	N/A
4 – 9	35	14	N/A	N/A
Below 9	50	20	1,500	600

*Depth below the proposed parkade elevation. The parkade is approximately 3 m below existing ground surface.

- c) Straight shaft or belled piles may be used for end bearing piles. In the case of belled piles, the bell diameter to shaft diameter ratio should not exceed 3:1, and the roof of the bell should not be sloped at more than 30° to the vertical.
- d) A minimum pile depth of 2.5 times the bell diameter has been assumed in calculating the above bearing capacity. If less cover is provided, the specified bearing capacity must be reduced.
- e) A minimum pile spacing of belled piles of 2.5 shaft diameters is recommended. In addition, edge to edge spacing of the bells of belled end bearing piles should be at least 0.5 m to minimize potential conflict during pile installations. Where bells are closer than one bell diameter, the adjacent piles in groups should not be excavated and poured until the initial pile base set up for at least 24 hours to prevent potential blow out and disturbance of the concrete.



- f) A minimum pile shaft diameter of 400 mm is recommended to prevent voids from forming during pouring of the concrete. Larger pile diameters are expected to be required due to casing requirements and also due to expected pile loads.
- g) Longitudinal reinforcement should be provided for the full pile length to resist potential uplift forces on the pile due to frost action or seasonal moisture variations. If piles are designed as tension elements, longitudinal reinforcing steel should extend into the pile bells, and the piles should be designed to resist the anticipated uplift stresses.
- h) All pile excavations should be thoroughly cleaned and visually inspected by qualified geotechnical personnel prior to pouring of the concrete to ensure a satisfactory base has been achieved. No slough or disturbed material should be allowed to remain in the pile excavations.
- i) Temporary steel casing(s) will be required during pile installation to extend the piles through the sand and granular layers encountered below the clay stratum and to prevent seepage and/or sloughing.
- j) Concrete should be poured immediately after drilling in order to reduce the potential for seepage and sloughing soil.

5.4.4 Continuous Flight Auger Piles

Concrete piles installed by Continuous Flight Augers (CFA), also referred to as the auger cast-in-place piles have been used in Alberta over the last ten or more years. CFA piles are essentially straight shaft piles that are installed by continuous flight auger techniques. The CFA augers provide temporary casing of the holes and are able to advance through mixed soil conditions including thick sand and gravel layers as encountered at this site. The currently available pile sizes locally range from about 300 mm to 1.2 m diameter with lengths of 20 m or greater.

CFA piles are expected to be a suitable pile type on a site wide basis and can be advanced through wet sands and gravel as expected at this site.

The piles are installed by advancing a continuous flight auger into the ground to the target depth to form the pile's diameter. Sand-cement grout or concrete is then pumped through the CFA hollow stem to fill the pile hole left as the auger is concurrently removed. After the auger is removed, reinforcement is inserted into the wet grout.



Further information on the design and installation of CFA piles is presented in a Geotechnical Engineering Circular (GEC) No. 8 – Design and Construction of Continuous Flight Auger Pile Foundations, published by The Federal Highway Administration (FHWA).

This type of pile cannot be readily inspected by down-hole inspection as with cast-in-place concrete piles and hence the successful installation relies heavily on the experience of the operator, and adequate testing of production piles.

CFA piles may be designed based on a combination of skin friction plus end bearing, as follows:

$$Q_T = Q_B + Q_s$$

Where

$$Q_T = \text{Ultimate static pile capacity (kN)}$$

$$Q_B = \text{Ultimate end bearing resistance (kN)}$$

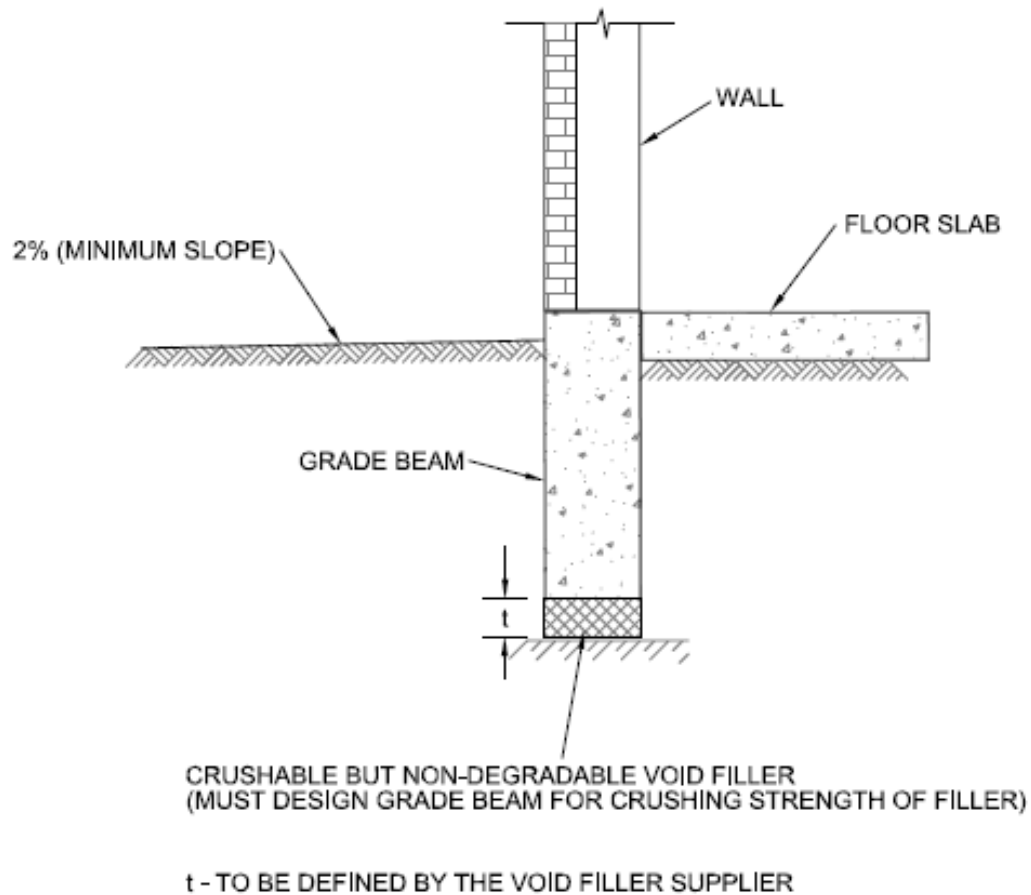
$$Q_s = \text{Ultimate shaft friction (kN)}$$

CFA piles may be designed using the ULS design parameters for compression and uplift as presented in Table 5.3.

5.5 Concrete Grade Beams

Piles used to support the building may require concrete grade beams and pile caps along the top of the piles. Precautions should be taken to prevent heaving of the grade beams due to seasonal moisture variation.

The recommended construction procedures for preventing heave under the grade beam are through use of a crushable non-degradable void form material (such as Beaver Plastics Frost Cushion) as shown in Figure 5.1. The grade beam must be designed in accordance with the crushing strength of the void filler used and the piles must be able to resist the resulting uplift load.



TYPICAL UNINSULATED GRADE BEAM
BASE OF GRADE BEAM ABOVE ZONE OF SEASONAL VOLUME CHANGE

FIGURE 5.1



5.6 Concrete Floor Slabs

The lowest level parkade slab may be founded on firm clay or loose sand at an approximate depth of about 3 m, which is expected to provide suitable subgrade support. A modulus of vertical subgrade reaction of 6 MPa/m may be used for structural design of a slab-on-grade founded on the prepared base as recommended above.

Soft materials within the floor footprint should be sub-excavated and replaced with clean granular material or fillcrete.

A minimum of 150 mm of clean, well-graded sand or gravel is recommended beneath floor slabs and along the outside of grade beams for leveling and drainage purposes. Coarse material greater than 50 mm in diameter should be avoided directly beneath the floor slab to prevent stress concentrations within the slab. The granular leveling course should be compacted to a uniform dry density of about 98 percent of SPMDD. A recommended typical gradation for free draining granular material, for use under the floor slabs (and also on the outside of the perimeter basement walls, if required) is provided below:

**TABLE 5.4
TYPICAL GRADATION FOR FREE DRAINING GRAVEL**

SIEVE	% PASSING
1 ½ (38,000 µm)	100
3/8 (10,000 µm)	65 - 100
No. 4 (5,000 µm)	50 - 90
No. 10 (2,000 µm)	35 - 75
No. 40 (400 µm)	10 - 45
No. 100 (150 µm)	0 - 20
No. 200 (75 µm)	0 - 5

Backfill around footings should consist of the select native sand placed in 150 mm lifts and be compacted to at least 98 percent SPMDD.

Other appropriate materials which fall outside the above recommended gradation limits may be suitable. Alternate materials should, however, be evaluated by a geotechnical engineer prior to placement.



5.7 Permanent Drainage

Based on the results of the groundwater level monitoring it is expected that the water level is below the parkade level. However, the possibility of perched water within the sand and gravel may provide a source of long term groundwater seepage.

Provision for permanent drainage should therefore be included in the design to prevent the build-up of hydrostatic pressure against the foundation walls and below the concrete floor slab-on-grade.

Drainage behind the permanent walls can be accomplished by installing granular drainage material or continuous geocomposite drainage material between the temporary shoring walls (i.e. lagging or shotcrete) and permanent walls. The drainage medium should extend down to the base of the perimeter footings and should be hydraulically connected to subdrains placed below floor slab level.

Backfilling between the lagging and walls (where required) should be a granular fill containing a maximum of 5 percent passing the 80 micron sieve and maximum particle size of 25 mm. The granular backfill should extend to within 1 m of the finished surface grade and should be compacted to about 95 percent of SPMDD. A 1 m thick cap of impervious clay should be placed over the granular fill at ground surface to reduce the ingress of surface water. Final grading should direct runoff away from the building.

5.8 Cement Type

Three tests were conducted to determine the water-soluble sulphate ion (SO_4) content of soil samples recovered from the boreholes. These tests showed the presence of 0.00 percent to 0.10 percent water-soluble sulphate ion content in the soil samples, indicating that there is no potential for sulphate attack on the subsurface concrete. As a result, CSA Type GU (General Use hydraulic cement) may be used in the subsurface concrete at this project site.

The recommendations stated above for the subsurface concrete at this site may require further additions and / or modifications due to structural, durability, service life or other considerations which are beyond the geotechnical scope.

In addition, if imported material is required to be used at the site and will be in contact with concrete, it is recommended that the fill soil be tested for sulphate content to determine whether the above stated recommendations remain valid.



5.9 Seismic Site Classification

The site classification as per Table 4.1.8.4.A of the National Building Code (2010) should be based on a 30 m deep test hole. As we do not have a 30 m deep test hole at the site, we reviewed the available in-house geotechnical information at this site along with the current investigation, which indicates that the bedrock may be encountered at depths of about 10 m. This information combined with data from our geotechnical investigation indicates that the site can be classified as Class C.

6. CONSTRUCTION INSPECTIONS

The performance of the buildings will depend upon the quality of workmanship during construction. This is particularly important in regard to foundation installations and other earthwork where variations in soil conditions could occur. Therefore, it is recommended that inspection be provided by qualified geotechnical personnel during foundation installation to confirm that they are installed in competent bearing material and that the stratigraphy is similar to those that have been assumed for the design. Compaction testing for backfill will also be required.

7. LIMITATIONS AND USE OF REPORT

There is a possibility that this report may form part of the design and construction documents for information purposes. This report was issued before any final design or construction details have been prepared or issued. Therefore, differences may exist between the report recommendations and the final design, the contract documents, or as observed during construction. In such instances, Thurber Engineering Ltd. should be contacted immediately to address these differences.

Designers and contractors undertaking or bidding the work should examine the factual results of the investigation, satisfy themselves to the adequacy of the information for design and construction, and make their own interpretation of the data as it may affect their proposed scope of work, cost, schedules, safety and equipment capabilities.

STATEMENT OF LIMITATIONS AND CONDITIONS

1. STANDARD OF CARE

This Report has been prepared in accordance with generally accepted engineering or environmental consulting practices in the applicable jurisdiction. No other warranty, expressed or implied, is intended or made.

2. COMPLETE REPORT

All documents, records, data and files, whether electronic or otherwise, generated as part of this assignment are a part of the Report, which is of a summary nature and is not intended to stand alone without reference to the instructions given to Thurber by the Client, communications between Thurber and the Client, and any other reports, proposals or documents prepared by Thurber for the Client relative to the specific site described herein, all of which together constitute the Report.

IN ORDER TO PROPERLY UNDERSTAND THE SUGGESTIONS, RECOMMENDATIONS AND OPINIONS EXPRESSED HEREIN, REFERENCE MUST BE MADE TO THE WHOLE OF THE REPORT. THURBER IS NOT RESPONSIBLE FOR USE BY ANY PARTY OF PORTIONS OF THE REPORT WITHOUT REFERENCE TO THE WHOLE REPORT.

3. BASIS OF REPORT

The Report has been prepared for the specific site, development, design objectives and purposes that were described to Thurber by the Client. The applicability and reliability of any of the findings, recommendations, suggestions, or opinions expressed in the Report, subject to the limitations provided herein, are only valid to the extent that the Report expressly addresses proposed development, design objectives and purposes, and then only to the extent that there has been no material alteration to or variation from any of the said descriptions provided to Thurber, unless Thurber is specifically requested by the Client to review and revise the Report in light of such alteration or variation.

4. USE OF THE REPORT

The information and opinions expressed in the Report, or any document forming part of the Report, are for the sole benefit of the Client. NO OTHER PARTY MAY USE OR RELY UPON THE REPORT OR ANY PORTION THEREOF WITHOUT THURBER'S WRITTEN CONSENT AND SUCH USE SHALL BE ON SUCH TERMS AND CONDITIONS AS THURBER MAY EXPRESSLY APPROVE. Ownership in and copyright for the contents of the Report belong to Thurber. Any use which a third party makes of the Report, is the sole responsibility of such third party. Thurber accepts no responsibility whatsoever for damages suffered by any third party resulting from use of the Report without Thurber's express written permission.

5. INTERPRETATION OF THE REPORT

- a) Nature and Exactness of Soil and Contaminant Description: Classification and identification of soils, rocks, geological units, contaminant materials and quantities have been based on investigations performed in accordance with the standards set out in Paragraph 1. Classification and identification of these factors are judgmental in nature. Comprehensive sampling and testing programs implemented with the appropriate equipment by experienced personnel may fail to locate some conditions. All investigations utilizing the standards of Paragraph 1 will involve an inherent risk that some conditions will not be detected and all documents or records summarizing such investigations will be based on assumptions of what exists between the actual points sampled. Actual conditions may vary significantly between the points investigated and the Client and all other persons making use of such documents or records with our express written consent should be aware of this risk and the Report is delivered subject to the express condition that such risk is accepted by the Client and such other persons. Some conditions are subject to change over time and those making use of the Report should be aware of this possibility and understand that the Report only presents the conditions at the sampled points at the time of sampling. If special concerns exist, or the Client has special considerations or requirements, the Client should disclose them so that additional or special investigations may be undertaken which would not otherwise be within the scope of investigations made for the purposes of the Report.
- b) Reliance on Provided Information: The evaluation and conclusions contained in the Report have been prepared on the basis of conditions in evidence at the time of site inspections and on the basis of information provided to Thurber. Thurber has relied in good faith upon representations, information and instructions provided by the Client and others concerning the site. Accordingly, Thurber does not accept responsibility for any deficiency, misstatement or inaccuracy contained in the Report as a result of misstatements, omissions, misrepresentations, or fraudulent acts of the Client or other persons providing information relied on by Thurber. Thurber is entitled to rely on such representations, information and instructions and is not required to carry out investigations to determine the truth or accuracy of such representations, information and instructions.
- c) Design Services: The Report may form part of design and construction documents for information purposes even though it may have been issued prior to final design being completed. Thurber should be retained to review final design, project plans and related documents prior to construction to confirm that they are consistent with the intent of the Report. Any differences that may exist between the Report's recommendations and the final design detailed in the contract documents should be reported to Thurber immediately so that Thurber can address potential conflicts.
- d) Construction Services: During construction Thurber should be retained to provide field reviews. Field reviews consist of performing sufficient and timely observations of encountered conditions in order to confirm and document that the site conditions do not materially differ from those interpreted conditions considered in the preparation of the report. Adequate field reviews are necessary for Thurber to provide letters of assurance, in accordance with the requirements of many regulatory authorities.

6. RELEASE OF POLLUTANTS OR HAZARDOUS SUBSTANCES

Geotechnical engineering and environmental consulting projects often have the potential to encounter pollutants or hazardous substances and the potential to cause the escape, release or dispersal of those substances. Thurber shall have no liability to the Client under any circumstances, for the escape, release or dispersal of pollutants or hazardous substances, unless such pollutants or hazardous substances have been specifically and accurately identified to Thurber by the Client prior to the commencement of Thurber's professional services.

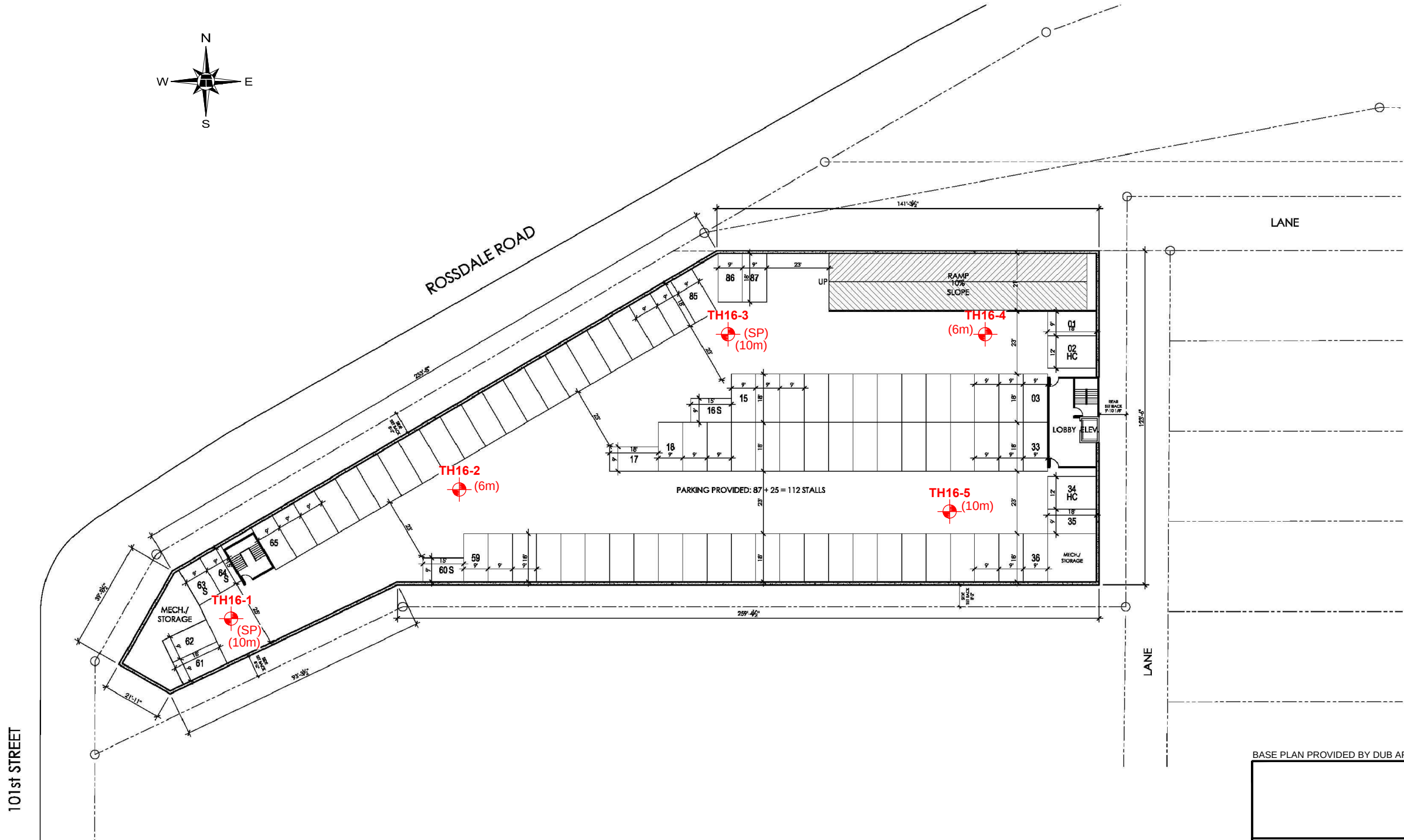
7. INDEPENDENT JUDGEMENTS OF CLIENT

The information, interpretations and conclusions in the Report are based on Thurber's interpretation of conditions revealed through limited investigation conducted within a defined scope of services. Thurber does not accept responsibility for independent conclusions, interpretations, interpolations and/or decisions of the Client, or others who may come into possession of the Report, or any part thereof, which may be based on information contained in the Report. This restriction of liability includes but is not limited to decisions made to develop, purchase or sell land.



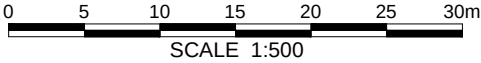
APPENDIX A

Drawing 12553-1 - Site Plan Showing Approximate Test Hole Locations



LEGEND

- APPROXIMATE TEST HOLE LOCATION
- (SP) STANDPIPE PIEZOMETER
- (10m) DEPTH OF THE TEST HOLE



BASE PLAN PROVIDED BY DUB ARCHITECTS LTD.



ROSSDALE CENTRE PHASE I
GEOTECHNICAL INVESTIGATION

SITE PLAN SHOWING APPROXIMATE
TEST HOLE LOCATIONS

DWG No. 12553-1

DRAWN BY	ML
DESIGNED BY	JGP
APPROVED BY	HSP
SCALE	APPROX. 1:500
DATE	APRIL 2016
FILE No.	12553



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APPENDIX B

Symbols and Terms Used on Test Hole Logs
Modified Unified Soils Classification
Test Hole Logs

SYMBOLS AND TERMS USED ON TEST HOLE LOGS

1. VISUAL TEXTURAL CLASSIFICATION OF MINERAL SOILS

<u>CLASSIFICATION</u>	<u>APPARENT PARTICLE SIZE</u>	<u>VISUAL IDENTIFICATION</u>
Boulders	Greater than 200 mm	Greater than 200 mm
Cobbles	75 mm to 200 mm	75 mm to 200 mm
Gravel	4.75 mm to 75 mm	5 mm to 75 mm
Sand	0.075 mm to 4.75 mm	Visible particles to 5 mm
Silt	0.002 mm to 0.075 mm	Non-Plastic particles, not visible to the naked eye
Clay	Less than 0.002 mm	Plastic particles, not visible to the naked eye

2. TERMS DESCRIBING CONSISTENCY (COHESIVE SOILS ONLY)

<u>DESCRIPTIVE TERM</u>	<u>APPROXIMATE UNDRAINED SHEAR STRENGTH</u>	<u>APPROXIMATE SPT * 'N' VALUE</u>
Very Soft	Less than 10 kPa	Less than 2
Soft	10 - 25 kPa	2 to 4
Firm	25 - 50 kPa	4 to 8
Stiff	50 - 100 kPa	8 to 15
Very Stiff	100 - 200 kPa	15 to 30
Hard	200 - 300 kPa	Greater than 30
Very Hard	Greater than 300 kPa	

} Modified from
National Building
Code

* SPT 'N' Value Standard Penetration Test 'N' Value - refers to the number of blows from a 63.5 kg hammer free falling a height of 0.76m to advance a standard 50mm outside diameter split spoon sampler for 0.3m depth into the undrilled portion of the test hole.

3. TERMS DESCRIBING DENSITY (COHESIONLESS SOILS ONLY)

<u>DESCRIPTIVE TERM</u>	<u>STANDARD PENETRATION TEST (SPT) (Number of Blows per 300 mm)</u>
Very Loose	0 - 4
Loose	4 - 10
Compact	10 - 30
Dense	30 - 50
Very Dense	Over 50

} Modified from
National Building
Code

4. LEGEND FOR TEST HOLE LOGS

SYMBOL FOR SAMPLE TYPE

■ Shelby Tube
▧ SPT
⊗ No Recovery

▨ A-Casing
▧ Grab
▩ Core

SYMBOLS USED FOR TEST HOLE LOGS

● MC - Moisture Content (% by weight) of soil sample
▼ Water Level
■ SPT Standard Penetration Test 'N' Value (Blows/300mm)
▲ CPen Shear Strength determined by pocket penetrometer
CVane Shear Strength determined by pocket vane
Cu Undrained Shear Strength determined by unconfined compression test
SO₄% Percent (%) of water soluble sulphate ions



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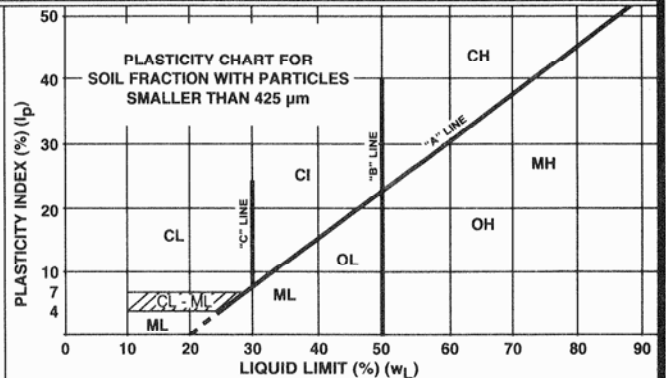
MODIFIED UNIFIED CLASSIFICATION SYSTEM FOR SOILS

(MODIFIED BY PFRA, 1985)

MAJOR DIVISION			GROUP SYMBOL	THURBER LOG SYMBOL	TYPICAL DESCRIPTION	LABORATORY CLASSIFICATION CRITERIA		
COARSE-GRAINED SOILS (MORE THAN HALF BY WEIGHT LARGER THAN 75µm)	GRAVELS MORE THAN HALF COARSE GRAINS LARGER THAN 4.75 mm	CLEAN GRAVELS (LITTLE OR NO FINES)	GW		WELL GRADED GRAVELS, GRAVEL - SAND MIXTURES, LITTLE OR NO FINES	Determine percentages of gravel and sand from grain size curve. Depending on percentages of fines (fraction smaller than 75µm) coarse grained soils are classified as follows: Less than 5% GW, GP, SW, SP More than 5% GM, GC, SM, SC Borderline cases requiring use of dual symbols	$C_u = \frac{D_{60}}{D_{10}} > 4$; $C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}} = 1 \text{ to } 3$	
			GP		POORLY GRADED GRAVELS, GRAVEL-SAND MIXTURES, LITTLE OR NO FINES		NOT MEETING ALL GRADATION REQUIREMENTS FOR GW	
		GRAVELS WITH FINES (APPRECIABLE AMOUNT OF FINES)	GM		SILTY GRAVELS, GRAVEL-SAND-SILT MIXTURES		ATTERBERG LIMITS BELOW "A" LINE I_p LESS THAN 4	Above "A" line with I_p between 4 and 7 are borderline cases requiring use of dual symbols
			GC		CLAYEY GRAVELS, GRAVEL-SAND-CLAY MIXTURES		ATTERBERG LIMITS ABOVE "A" LINE I_p MORE THAN 7	
	SANDS MORE THAN HALF COARSE GRAINS SMALLER THAN 4.75 mm	CLEAN SANDS (LITTLE OR NO FINES)	SW		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		POORLY GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES		NOT MEETING ALL GRADATION REQUIREMENTS FOR SW	
		SAND WITH FINES (APPRECIABLE AMOUNT OF FINES)	SM		SILTY SANDS, SAND-SILT MIXTURES		ATTERBERG LIMITS BELOW "A" LINE I_p LESS THAN 4	Above "A" line with I_p between 4 and 7 are borderline cases requiring use of dual symbols
			SC		CLAYEY SANDS, SAND-CLAY MIXTURES		ATTERBERG LIMITS ABOVE "A" LINE I_p 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		INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTY OR CLAYEY FINE SANDS OR CLAYEY SILTS WITH SLIGHT PLASTICITY	CLASSIFICATION IS BASED UPON PLASTICITY CHART (see below)		
		$w_L > 50\%$	MH		INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS, FINE SANDY OR SILTY SOILS			
	CLAYS ABOVE "A" LINE NEGLECTIBLE ORGANIC CONTENT	$w_L < 30\%$	CL		INORGANIC CLAYS OF LOW PLASTICITY, GRAVELLY, SANDY, OR SILTY CLAYS, LEAN CLAYS			
		$30\% < w_L < 50\%$	CI		INORGANIC CLAYS OF MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS			
		$w_L > 50\%$	CH		INORGANIC CLAYS OF HIGH PLASTICITY, FAT CLAYS			
	ORGANIC SILTS & CLAYS BELOW "A" LINE	$w_L < 50\%$	OL		ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW AND MEDIUM PLASTICITY			
		$w_L > 50\%$	OH		ORGANIC CLAYS OF HIGH PLASTICITY, ORGANIC SILTS			
	HIGHLY ORGANIC SOILS		Pt		PEAT AND OTHER HIGHLY ORGANIC SOILS		STRONG COLOR OR ODOR, AND OFTEN FIBROUS TEXTURE	

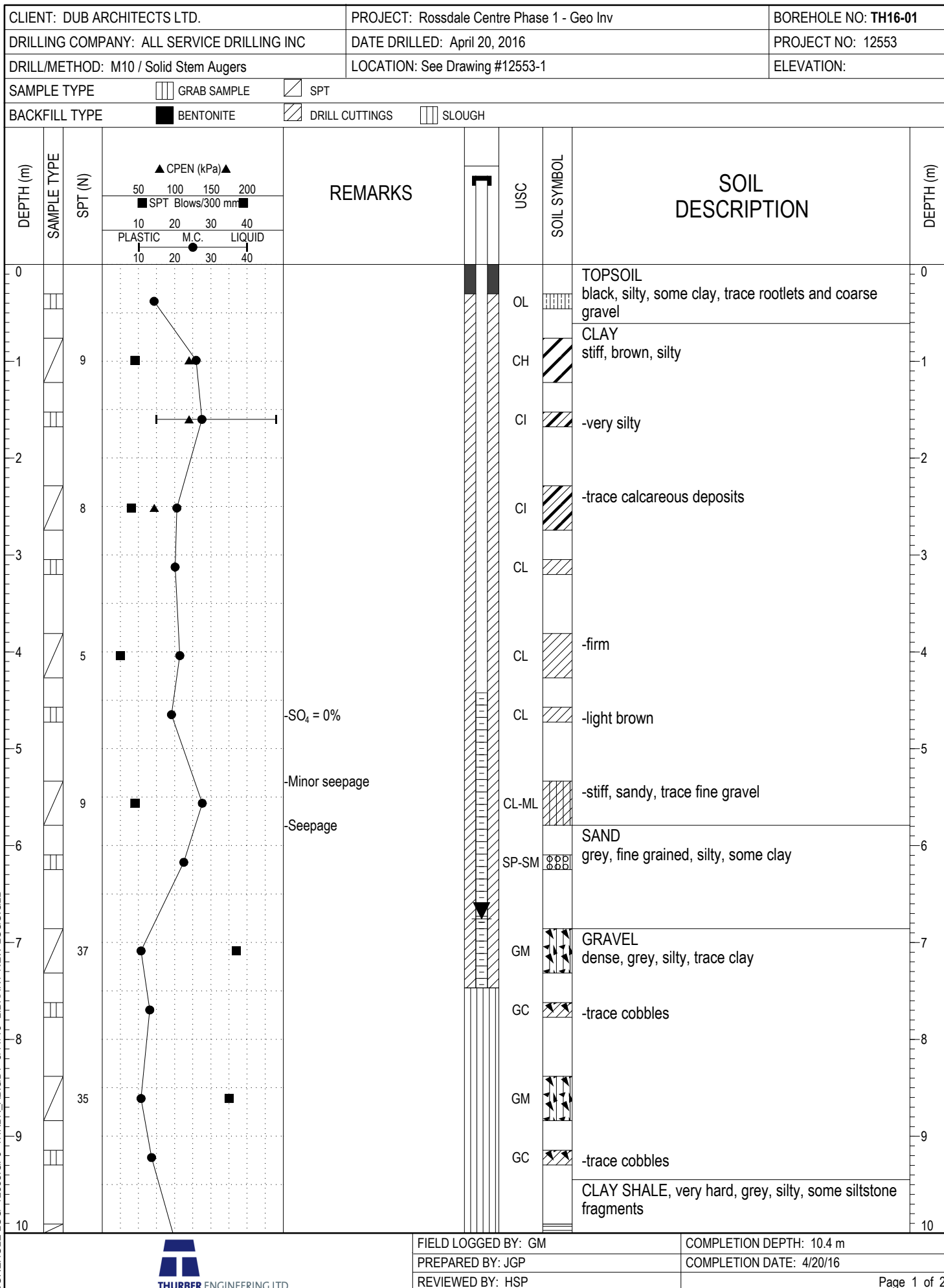
SPECIAL SYMBOLS

	BEDROCK (UNDIFFERENTIATED)		OVERBURDEN (UNDIFFERENTIATED)
	SANDSTONE		SILTSTONE
	CLAYSTONE (CLAYSHALE OR MUDSTONE)		
	LIMESTONE		
	CONGLOMERATE		
	COAL		



THURBER ENGINEERING LTD.

**MODIFIED
UNIFIED CLASSIFICATION SYSTEM
FOR SOILS**
(MODIFIED BY PFRA, 1985)



CLIENT: DUB ARCHITECTS LTD.			PROJECT: Rossdale Centre Phase 1 - Geo Inv			BOREHOLE NO: TH16-01		
DRILLING COMPANY: ALL SERVICE DRILLING INC			DATE DRILLED: April 20, 2016			PROJECT NO: 12553		
DRILL/METHOD: M10 / Solid Stem Augers			LOCATION: See Drawing #12553-1			ELEVATION:		
SAMPLE TYPE			☐ GRAB SAMPLE ☒ SPT					
BACKFILL TYPE			☒ BENTONITE ☒ DRILL CUTTINGS ☐ SLOUGH					

DEPTH (m)	SAMPLE TYPE	SPT (N)	▲ CPEN (kPa) ▲ 50 100 150 200 ■ SPT Blows/300 mm ■ 10 20 30 40 PLASTIC M.C. LIQUID 10 20 30 40	REMARKS	SLOTTED PIEZOMETER	USC	SOIL SYMBOL	SOIL DESCRIPTION	DEPTH (m)
10		52		>>■ Cpen > 215kPa		CS		CLAY SHALE - CONTINUED	10
11								END OF TEST HOLE AT 10.4m UPON COMPLETION: (Below ground surface) -Slough at 7.5m Standpipe piezometer installed WATER LEVEL BELOW GROUND SURFACE: -April 25, 2016 = 6.8m	11
12									12
13									13
14									14
15									15
16									16
17									17
18									18
19									19
20									20

 THURBER ENGINEERING LTD.	FIELD LOGGED BY: GM	COMPLETION DEPTH: 10.4 m
	PREPARED BY: JGP	COMPLETION DATE: 4/20/16
	REVIEWED BY: HSP	

CLIENT: DUB ARCHITECTS LTD.		PROJECT: Rossdale Centre Phase 1 - Geo Inv		BOREHOLE NO: TH16-02			
DRILLING COMPANY: ALL SERVICE DRILLING INC		DATE DRILLED: April 20, 2016		PROJECT NO: 12553			
DRILL/METHOD: M10 / Solid Stem Augers		LOCATION: See Drawing #12553-1		ELEVATION:			
SAMPLE TYPE ☐ GRAB SAMPLE ☒ NO RECOVERY ☒ SPT							
DEPTH (m)	SAMPLE TYPE	SPT (N)	REMARKS	USC	SOIL SYMBOL	SOIL DESCRIPTION	DEPTH (m)
0						TOPSOIL	0
1		4				CLAY soft, brown, silty	1
2							2
3		5				-firm, sandy	3
4		4				-light grey, sandy, trace calcareous deposits	4
5						-some high plastic clay	5
6		4	-Seepage			SAND loose, grey, fine grained, silty, trace clay	6
7						-interbedded with clay layers	7
8						END OF TEST HOLE AT 6.9m UPON COMPLETION: (Below ground surface) -Slough at 6.4m -Water at 6.2m Backfilled with drill cuttings and bentonite chips at surface	8
9							9
10							10



THURBER ENGINEERING LTD.

FIELD LOGGED BY: GM

PREPARED BY: JGP

REVIEWED BY: HSP

COMPLETION DEPTH: 6.9 m

COMPLETION DATE: 4/20/16

Page 1 of 1

CLIENT: DUB ARCHITECTS LTD.		PROJECT: Rosedale Centre Phase 1 - Geo Inv		BOREHOLE NO: TH16-03	
DRILLING COMPANY: ALL SERVICE DRILLING INC		DATE DRILLED: April 20, 2016		PROJECT NO: 12553	
DRILL/METHOD: M10 / Solid Stem Augers		LOCATION: See Drawing #12553-1		ELEVATION:	
SAMPLE TYPE ☐ GRAB SAMPLE ☒ SPT					
BACKFILL TYPE ☒ BENTONITE ☒ DRILL CUTTINGS ☐ SLOUGH					

DEPTH (m)	SAMPLE TYPE	SPT (N)	REMARKS	USC	SOIL SYMBOL	SOIL DESCRIPTION	DEPTH (m)
0					OL	TOPSOIL, black, silty, some rootlets	0
1		6			CH	CLAY firm, brown, silty, trace calcareous deposits	1
2			-SO ₄ = 0%		CI		2
3		5			CI		3
4		4			CL	-grey, some fine sand	4
5					SM	SAND loose, grey, fine grained, silty	5
6		8			SM	-trace coal deposits	6
7		36	-Seepage		SP-SM	-some clay	7
8					SP-SM	-dark grey, trace coal deposits	8
9		48			GP-GM	GRAVEL dense, brown, silty, trace cobbles	9
10					SP	Sand	10
					SP		
					GM	-trace coal fragments and clay	
						SANDSTONE, dense, grey, fine grained, clayey to some clay	

		FIELD LOGGED BY: GM PREPARED BY: JGP REVIEWED BY: HSP	COMPLETION DEPTH: 10.4 m COMPLETION DATE: 4/20/16
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CLIENT: DUB ARCHITECTS LTD.			PROJECT: Rossdale Centre Phase 1 - Geo Inv			BOREHOLE NO: TH16-03		
DRILLING COMPANY: ALL SERVICE DRILLING INC			DATE DRILLED: April 20, 2016			PROJECT NO: 12553		
DRILL/METHOD: M10 / Solid Stem Augers			LOCATION: See Drawing #12553-1			ELEVATION:		
SAMPLE TYPE			☐ GRAB SAMPLE ☒ SPT					
BACKFILL TYPE			☒ BENTONITE ☒ DRILL CUTTINGS ☐ SLOUGH					

DEPTH (m)	SAMPLE TYPE	SPT (N)	▲ CPEN (kPa) ▲ 50 100 150 200 ■ SPT Blows/300 mm ■ 10 20 30 40 PLASTIC M.C. LIQUID 10 20 30 40	REMARKS	SLOTTED PIEZOMETER	USC	SOIL SYMBOL	SOIL DESCRIPTION	DEPTH (m)
10		35				SS		SANDSTONE - CONTINUED	10
11								END OF TEST HOLE AT 10.4m UPON COMPLETION: (Below ground surface) -Slough at 7.2m -Water at 7.0m Standpipe piezometer installed WATER LEVEL BELOW GROUND SURFACE: -April 25, 2016 = 7.2m	11
12									12
13									13
14									14
15									15
16									16
17									17
18									18
19									19
20									20

 THURBER ENGINEERING LTD.	FIELD LOGGED BY: GM	COMPLETION DEPTH: 10.4 m
	PREPARED BY: JGP	COMPLETION DATE: 4/20/16
	REVIEWED BY: HSP	Page 2 of 2

CLIENT: DUB ARCHITECTS LTD.		PROJECT: Rossdale Centre Phase 1 - Geo Inv		BOREHOLE NO: TH16-04	
DRILLING COMPANY: ALL SERVICE DRILLING INC		DATE DRILLED: April 20, 2016		PROJECT NO: 12553	
DRILL/METHOD: M10 / Solid Stem Augers		LOCATION: See Drawing #12553-1		ELEVATION:	
SAMPLE TYPE ☐ GRAB SAMPLE ☒ SPT					

DEPTH (m)	SAMPLE TYPE	SPT (N)	▲ CPEN (kPa)▲ 50 100 150 200 ■ SPT Blows/300 mm■ 10 20 30 40 PLASTIC M.C. LIQUID 10 20 30 40	REMARKS	USC	SOIL SYMBOL	SOIL DESCRIPTION	DEPTH (m)
0					OL		TOPSOIL black, silty, trace silt, clay, and rootlets	0
1		7			CH		CLAY firm, light brown, silty, trace calcareous deposits	1
2					CI			2
3		4			CI			3
4		4			CL-CI			4
5					CL-CI			5
6		7			CL		-some silt and clay	6
7					SM		SAND brown, very fine grained, silty, some clay	6
7				-Seepage			END OF TEST HOLE AT 6.9m UPON COMPLETION: (Below ground surface) -No slough -Water at 6.9m Backfilled with drill cuttings and bentonite chips at surface	7
8								8
9								9
10								10

 THURBER ENGINEERING LTD.	FIELD LOGGED BY: GM	COMPLETION DEPTH: 6.9 m
	PREPARED BY: JGP	COMPLETION DATE: 4/20/16
	REVIEWED BY: HSP	Page 1 of 1

CLIENT: DUB ARCHITECTS LTD.		PROJECT: Rossdale Centre Phase 1 - Geo Inv		BOREHOLE NO: TH16-05	
DRILLING COMPANY: ALL SERVICE DRILLING INC		DATE DRILLED: April 20, 2016		PROJECT NO: 12553	
DRILL/METHOD: M10 / Solid Stem Augers		LOCATION: See Drawing #12553-1		ELEVATION:	
SAMPLE TYPE ☐ GRAB SAMPLE ☒ NO RECOVERY ☐ SPT					
BACKFILL TYPE ☒ BENTONITE ☐ DRILL CUTTINGS ☐ SLOUGH					

DEPTH (m)	SAMPLE TYPE	SPT (N)	REMARKS	USC	SOIL SYMBOL	SOIL DESCRIPTION	DEPTH (m)
0						TOPSOIL	0
				CI		CLAY stiff, light brown, silty	
1		8					1
				CI		-trace fine sand	
2							2
		6		CI		-firm, trace calcareous deposits	
3				CI		-brown	3
				CI		-trace oxides	
4		5					4
				CI		-some oxides and calcareous deposits	
5							5
		7		SC		Sand loose	
6				CL			6
			-Seepage				
7		30		GC		GRAVEL dense, brown, silty, trace clay	7
				GC		-trace cobbles	
8							8
		54	>>	SP		Sand very dense	
9				GC		-trace cobbles and boulders	9
10							10

 THURBER ENGINEERING LTD.	FIELD LOGGED BY: GM	COMPLETION DEPTH: 10.4 m
	PREPARED BY: JGP	COMPLETION DATE: 4/20/16
	REVIEWED BY: HSP	

BOREHOLE LOG 12553.GPJ THPBR AB GDT 5/11/16 - LIBRARY-NEW LOGO.GLB

CLIENT: DUB ARCHITECTS LTD.		PROJECT: Rossdale Centre Phase 1 - Geo Inv		BOREHOLE NO: TH16-05	
DRILLING COMPANY: ALL SERVICE DRILLING INC		DATE DRILLED: April 20, 2016		PROJECT NO: 12553	
DRILL/METHOD: M10 / Solid Stem Augers		LOCATION: See Drawing #12553-1		ELEVATION:	
SAMPLE TYPE		☐ GRAB SAMPLE ☒ NO RECOVERY ☒ SPT			
BACKFILL TYPE		☒ BENTONITE ☒ DRILL CUTTINGS ☐ SLOUGH			

DEPTH (m)	SAMPLE TYPE	SPT (N)	■ SPT Blows/300 mm ■ 10 20 30 40 PLASTIC M.C. LIQUID 10 20 30 40	REMARKS	SLOTTED PIEZOMETER	USC	SOIL SYMBOL	SOIL DESCRIPTION	DEPTH (m)
10		28	● ■			GC		GRAVEL - CONTINUED -compact END OF TEST HOLE AT 10.4m UPON COMPLETION: (Below ground surface) -Slough at 8.1m -Water at 7.0m Standpipe piezometer installed WATER LEVEL BELOW GROUND SURFACE: -April 25, 2016 = 6.9m	10
11									11
12									12
13									13
14									14
15									15
16									16
17									17
18									18
19									19
20									20

 THURBER ENGINEERING LTD.	FIELD LOGGED BY: GM	COMPLETION DEPTH: 10.4 m
	PREPARED BY: JGP	COMPLETION DATE: 4/20/16
	REVIEWED BY: HSP	



APPENDIX C

Recommended Construction Procedures



RECOMMENDED CONSTRUCTION PROCEDURES

The following construction procedures are considered to represent good practice and are to be read in conjunction with the text of this report.

1. PROOF ROLLING

- 1.1 Proof rolling is a method of detecting soft areas in a subgrade for fill, pavement, floors or foundations. The intent is to detect softened areas not revealed by the test holes or visual examination of the site surface, and is used where normal scarification and compacting procedures would not be successful in detecting and eliminating soft areas. It is usually accomplished with the use of heavy 130 to 220 kN (15-25 ton) compaction equipment with high contact wheel pressures on independent axles, although heavily loaded single axle trucks will provide the equivalent result.
- 1.2 The procedure requires 2 complete passes with the heavy equipment in one direction and then a second series of 2 passes made at right angles to the first series.
- 1.3 While the passes are being made, any softened, rutted or displaced areas detected should be examined and either recompact with additional fill or the existing material removed and replaced with better quality material.

2. EXCAVATED FOUNDATIONS

- 2.1 Excavation close to foundation level should be done carefully to avoid disturbance of the soil. It is essential to prevent the soil at foundation level from deterioration due to excessive drying or becoming wet from surface or seepage water. Good drainage both during and after construction is essential.
- 2.2 Sumps, if required, should be located well away from the foundation area. Softened or overdried soil must be removed and replaced by lean mix concrete or by extending the foundations.
- 2.3 The foundation must be kept from freezing both during and after construction. Foundation concrete should not be placed on or against frozen soil.



3. BACKFILLING

- 3.1 Backfill around foundations should be placed in such a manner so as to prevent settlement and to be relatively impervious near the surface so that water does not pond against foundations nor be allowed to seep into the soil.
- 3.2 Backfill should not be placed until the structure has sufficient strength to withstand the earth pressures resulting from placement and compaction.
- 3.3 All backfill around grade beams, foundation walls, etc. must be carefully and uniformly compacted. The backfill should be placed in even layers and no frozen or organic material should be incorporated into the fill. All lumps of material must be broken down or squeezed together during placing and compaction.
- 3.4 The final grade (allowing for some settlement of the backfill) should shed water away from the structure.
- 3.5 During construction, precautions should be taken to prevent water ponding in grade beam excavations thereby acting as a source of water to soften the soil under the floor slab area or providing a source of water for frost action if the building is not heated during freezing weather.

4. BORED CAST-IN-PLACE CONCRETE PILES

- 4.1 If there is evidence of water bearing and/or sloughing soil, casing should be used to seal off the water or prevent the sloughing of the sides of the hole. The concrete and reinforcing steel should be on hand and placed as soon as the pile hole has been completed and approved.
- 4.2 Pile bells, if used, should be formed entirely in self-supporting soil and it may be necessary in some cases to extend the pile bell if caving occurs at the location of the bell.
- 4.3 Water should not be left ponded on the pile base and should be removed, or dried by the use of dry cement when permitted by the engineer.
- 4.4 Concrete should be placed without segregation and carefully vibrated throughout the full length of the pile to ensure that voids do not exist in the pile shaft. The concrete slump should be between 75 and 125 mm with a minimum compressive strength at 28 days of



21 MPa (3000 psi). Higher compressive strengths may be required for structural or durability reasons, and higher slumps may be necessary for closely spaced reinforcing bars or where concrete is to be tremied under water.

- 4.5 Steel reinforcing should be tied into the grade beam reinforcing steel. This recommendation is important where the soil below grade beam can swell from a change in moisture content or by frost action before the building is heated.
- 4.6 Piles closer than 2 1/2 diameters should not be drilled and poured consecutively unless permitted by the engineer and depending upon soil conditions. Where the drilling operation might affect the concrete in the adjacent pile, the drilling should not be carried out until the concrete has at least 24 hours to set, or before the concrete has reached its initial set.

Geotechnical Investigation
Rossdale Centre Multi-Family Development
9803 and 9806 to 9842 – 100 Street NW
Edmonton, Alberta

Prepared For:
Five Oaks Inc.

File No. 1-22776
July 2021

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Geotechnical

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Materials

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Radon

Materials Testing

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APPENDIX I	Standard Terms and Conditions
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1.0 INTRODUCTION

Shelby Engineering Ltd. (Shelby) has completed a Geotechnical Investigation for the proposed Rossdale Centre Multi-family development to be located in Edmonton, Alberta.

The scope of the investigation detailed herein was provided in our proposal (#19980) dated May 20, 2021. Mr. Norman Kloeber of Dub Architects authorized this investigation on behalf of Five Oaks Inc. on May 27, 2021.

2.0 DESCRIPTION OF SITE AND PROPOSED DEVELOPMENT

The site for the proposed development comprises three parcels (Parcels A to C) totaling 4 hectares of land. These parcels are located south of Rossdale Road NW, and west (Parcels A and B) or east (Parcel C) of 100 Street NW, as shown on the site location plan enclosed as Figure 1, Appendix II.

The legal descriptions and municipal addresses of the parcels are presented below:

Table 2.1: Legal Descriptions and Municipal Addresses

Parcel	Legal Description	Municipal Address
A	Lot 2, Block 2, Plan 0021114	9856 - 100 Street NW
B	Lot 28, Block 7, Plan Q Lot 29, Block 7, Plan Q Lot 30, Block 7, Plan Q Lot 26, Block 7, Plan Q Lot 27, Block 7, Plan Q	9806 - 100 Street NW
	Lot 25, Block 7, Plan Q	9818 - 100 Street NW
	Lot 24, Block 7, Plan Q	9822 - 100 Street NW
	Lot 23, Block 7, Plan Q	9826 - 100 Street NW
	Lot 22, Block 7, Plan Q	9828 - 100 Street NW
	Lot 21, Block 7, Plan Q	9832 - 100 Street NW
	Lot 20, Block 7, Plan Q	9834 - 100 Street NW
	Lot 19, Block 7, Plan Q	9838 - 100 Street NW
	Lot 18, Block 7, Plan Q	9842 - 100 Street NW

Table 2.1: Legal Descriptions and Municipal Addresses

Parcel	Legal Description	Municipal Address
C	Lot 2, Block 1, Plan 0021114	9803 - 100 Street NW
	Lot 14, Block 15, Plan Q	9843 - 100 Street NW
	Lot 13, Block 15, Plan Q	
	Lot 15, Block 15, Plan Q	
	Lot 12, Block 15, Plan Q	

The proposed development is expected to comprise three 15-storey multi-family residential buildings (one on each parcel) each with one level of underground parkade. It is assumed that asphalt pavement areas will be developed around the proposed buildings.

A review of historical air photos, available in our library, indicates that the site was previously developed with several buildings.

3.0 INVESTIGATION METHODOLOGY

The field investigation, supervised by Shelby staff, was conducted on June 9, 2021 and entailed drilling six test holes using a truck mounted drill rig equipped with solid stem augers. The test holes were drilled to depths ranging from 4.25m to 11.90m below existing grade within the proposed building footprints in Parcels B and C. The test holes were advanced at the locations shown on the test hole location plan, enclosed as Figure 2, Appendix II.

For Parcel A, a geotechnical investigation was completed by others (Thurber Engineering Ltd.) in 2016 for a previous concept plan comprising a 4-storey building with a one-level underground parkade. Test hole logs and site plan from Thurber's 2016 report are enclosed in Appendix III.

Disturbed soil samples were obtained at 300mm below existing grade and thereafter at regular depth intervals of 0.75m. Standard Penetration Tests (SPTs) were conducted at selected depth intervals.

All samples were returned to our laboratory for visual classification and determination of insitu moisture content. Additional laboratory tests to determine Atterberg limits and concentrations of water-soluble sulphate salts were also conducted on select samples.

3.1 SUBSURFACE CONDITIONS

The general stratigraphy encountered at the test hole locations was comprised of surficial fill underlain by clay, silt and/or sand followed by glacial till.

The consistency and extent of the various soil strata evidenced at test hole locations will vary between the test hole locations and in areas of the site that have not been explored.

All depths indicated below are referenced to surface grades as existed at the time of this investigation.

Test hole logs detailing the subsurface conditions and laboratory test results are enclosed as Figures 3 to 8, Appendix II.

3.1.1 Fill

Surficial fill was encountered at all test hole locations and extended to depths ranging from 1.2m to 2.3m below grade.

The fill was generally comprised of surficial topsoil underlain by clay and/or silt. The clay fill was generally described as silty, damp to moist except for wet conditions in one test hole (TH-1), firm to stiff and medium to high plastic. The silt fill was generally described as moist, firm and low plastic. The fill contained trace to some organics, as well as concrete/brick debris in some test holes.

The fill is likely composed of disturbed material native to the area placed during site grading activities and/or material used to backfill former basements of the previously demolished buildings. Details pertaining to placement of the fill, including compaction and moisture control, are not known.

Differing thickness of fill/organics may be present in areas of the site between and apart from the test hole locations. As such, actual stripping volumes may differ from those calculated using the thicknesses encountered at the test hole locations. To mitigate potential discrepancies, additional investigation using shallow test pits could be considered.

3.1.2 Clay/Silt

Interbedded layers of clay and/or silt were encountered beneath the fill at all test hole locations and extended to depths ranging from 3.55m to 6.4m below grade. The clay had a high silt content and was generally described as low to medium plastic, soft to firm and damp to moist except for some wet to saturated conditions at depths starting from 4.3m to 6.2m below grade in four test holes. The silt was soft to firm, damp to moist and low plastic with soft to firm consistency.

3.1.3 Sand

Sand was encountered beneath the clay or silt at the deep test hole locations (TH-1 to TH-4) and extended to depths ranging from 7.3m to 9.0m below grade. The sand was generally described as fine grained and damp to moist except for saturated conditions starting from 6.0m to 6.5m below grade. The sand had variable silt and/or gravel content and generally had a loose relative density.

3.1.4 Glacial Till

Glacial till, comprised of sand till and/or clay till, was encountered beneath the sand at each deep test hole location, and extended to the maximum depth of drilling (i.e., at least 11.90m below grade). The sand till was described as silty, low plastic, moist and hard. The clay till was described as silty, sandy, moist and hard with medium to high plasticity. Interbedded coal and/or clay shale layers were encountered within the till.

Although not visually confirmed, cobbles or boulders are typically randomly present within glacial till deposits and may be present within the clay till matrix on the subject site.

3.2 GROUNDWATER AND SLOUGHING OBSERVATIONS

Significant seepage and sloughing conditions were encountered during fieldwork.

The table below provides a summary of slough and groundwater measurements taken upon completion of drilling, and groundwater measurements taken 12 days after standpipe installation (each relative to surface grades at the time of drilling). Slough and groundwater measurements are also recorded on the test hole logs enclosed in Appendix II.

Table 3.1: Slough and Groundwater Accumulations

Test Hole Location	Depth Below Grade (m)		
	On Completion of Drilling		Water Level After 12 Days
	Slough	Water	
TH-1	7.50	7.50	7.55
TH-2	6.70	6.70	--
TH-3	6.70	6.70	--
TH-4	9.75	9.75	--
TH-5	None	Dry	Dry (to 4.25)
TH-6	None	Dry	Dry (to 4.25)

Based on these readings, the current stabilized groundwater table on the subject site appears to be at a depth of approximately 7.5m from grade. However, based on the wet to saturated subsurface soil conditions noted at shallower depths during drilling, groundwater seepage from perched groundwater is expected at depths below approximately 4.5m in some areas of the site.

Groundwater levels fluctuate seasonally and in response to precipitation. Variation on the order of 1m or more is possible within any given year, with higher groundwater levels expected in spring and summer months. As such, different groundwater levels may be encountered at the time of construction.

4.0 RECOMMENDATIONS

The following sections contain recommendations for Parcels B and C related to deep foundations, site grading and building pad preparation, subsurface utility installation, floor slab support, lateral earth pressures for parkade design, and the design of asphalt pavements. Mixing of different foundation types is not recommended.

Unless otherwise noted, the recommendations assume that the final exterior grades for the proposed development will be coincident with surface grades existing at the time of this investigation, and that the parkades will be established approximately 3.0m below final exterior grades. Appropriate adjustments must be made to the referenced depths/elevations with consideration to any difference between the referenced grades and final grades. Shelby must be advised of any grade changes so that we may assess the effect, if any, of grade changes on the recommendations.

Supplementary recommendations and comments related to Parcel A are presented in Section 5.0.

4.1 DEEP FOUNDATIONS

The following deep foundation types are believed to be feasible at this site considering the Geotechnical conditions encountered.

4.1.1 Cast-in-Place Concrete Skin Friction Piles

Cast-in-place concrete friction piles may be designed using the following skin friction parameters:

Table 4.1: Cast-in-Place Concrete Skin Friction Pile Design Parameters

Depth Below Grade (m)	Ultimate Skin Friction (kPa)	Factored ULS Skin Friction (kPa)
0.0 to 1.5	0	0
1.5 to 8.0	22	9
Below 8.0	75	30

The factored ULS skin friction values above include a geotechnical resistance factor of 0.4 (for resistance to axial compressive load) applied to the ultimate skin friction. For assessing resistance to structural uplift loads, a geotechnical resistance factor of 0.3 must be applied to the ultimate skin friction values to calculate the appropriate factored ULS friction values.

The serviceability limit state (SLS) is not applicable to skin friction piles in most cases.

In heated structures, skin friction resistance along the upper 1.5m of pile length (or the portion of the pile in contact with fill, if greater) should be ignored. In unheated areas the skin friction resistance along the upper 2.3m of the pile length (or the portion of the pile in contact with fill, if greater) should be ignored.

A minimum shaft diameter of 400mm is recommended. The minimum clear spacing (edge to edge) of any two adjacent skin friction piles should be a distance equal to the sum of the diameters of those piles. Closer spacing would require a reduction in the factored shaft resistance provided above. Shelby should be contacted to provide revised values, if necessary.

Reinforcement within the pile should be sufficient to prevent adverse effects from seasonal frost penetration or moisture content variations. For piles subjected to frost action during construction, the reinforcement should extend to a minimum length of 6m below the lowest elevation of the structure exposed to freezing conditions.

Concrete should be placed as soon as possible after drilling of the pile excavation. The upper 3m of concrete should be vibrated to consolidate the concrete at the top of the pile.

Significant groundwater seepage and sloughing conditions were encountered during fieldwork. Casing should be on site and utilized as necessary to prevent seepage or sloughing from having a detrimental effect on the pile installation.

The Alberta Building Code specifies full time continuous field review, by a suitably qualified individual, during installation of all deep foundation elements.

4.1.2 Cast-in-Place Concrete Belled End Bearing Piles

Cast-in-place concrete belled end bearing piles founded within hard clay till (and not in sand/sand till or gravel) may be designed using the following parameters:

Table 4.2: Cast-in-Place Concrete End Bearing Pile Design Parameters

Minimum Belling Depth Below Grade (m)	Ultimate Base Resistance (kPa)	Factored ULS Base Resistance (kPa)	SLS Base Resistance (kPa)
10.0	2250	900	750

The factored ULS base resistance above includes a geotechnical resistance factor of 0.4 (for resistance to axial compressive load) applied to the ultimate values. Shelby should be contacted to provide additional recommendations if belled piles will be subjected to structural uplift loads.

The SLS bearing pressure above is based on a typical pile settlement of approximately 1.0% of the bell diameter at the working structural loads. If the required bell diameters result in a predicted pile settlement that is higher than acceptable for the project, Shelby should be contacted to provide additional guidance.

For both ULS and SLS cases, the applicable bearing pressure should be compared to the corresponding structural loads to determine the foundation size required to satisfy both conditions.

The base resistance values above assume a depth of cover of 2.5 times the bell diameter. Lower base resistance values would be required if less cover is provided. Shelby should be contacted to provide revised values, if necessary.

A minimum shaft diameter of 400mm is recommended. The bell diameter should not exceed 3 times the shaft diameter. The effect of skin friction should not be included in the design of belled, end-bearing piles on this site.

Center-to-center spacing of adjacent belled piles should be a minimum of the sum of the bell diameters of the piles being considered. If the minimum pile spacing cannot be accommodated, Shelby should be contacted as closer spacing may require either a revision to the parameters provided above or a possible staggering of the belling depths. When closely spaced belled piles founded at the same elevation are used, a minimum edge to edge bell spacing of 300mm is recommended.

Bell formation may be difficult within the till stratum. Interested piling contractors should consider attempting proof-of-concept test piles to satisfy themselves of installation feasibility and equipment requirements.

The base of all piles utilizing end bearing resistance must be dry and thoroughly cleaned of all loosened material, build-up or slough. Following drilling and cleaning, pile excavations should be inspected to ensure that an adequate bearing surface has been prepared.

Reinforcement within the pile should be sufficient to prevent adverse effects from seasonal frost penetration or moisture content variations. Where the bell is required to resist uplift forces (including from frost adfreeze during or after construction), the reinforcement should extend to near the base of the bell.

Significant groundwater seepage and sloughing conditions were encountered during fieldwork. Casing should be on site and utilized as required to prevent seepage or sloughing from having a detrimental effect on the pile installation. Bells cannot be formed within sloughing layers, such as sand/sand till or gravel. This may require field alteration of bell elevation or altering of the pile type.

Concrete should be placed as soon as possible after drilling and inspection of the pile excavation to reduce the risk of seepage or sloughing from having a detrimental effect on the pile installation. The upper 3m of concrete should be vibrated to consolidate the concrete near the top of the pile.

The Alberta Building Code specifies full time continuous field review, by a suitably qualified individual, during installation of all deep foundation elements.

4.1.3 Pile Load Test Considerations

For projects with large and/or many piles, a static pile load test can have a significant positive impact on the foundation costs. For the purposes of assessing the budgetary impacts of conducting a static pile load test for this project, the factored ULS values presented herein may be increased by a factor of 1.5 (representing the use of a geotechnical resistance factor of 0.6) based on the successful completion of the pile load test. A test pile instrumented with strain gauges may also allow for an increase in the underlying ultimate values (upon which the resistance factor is then applied), if the test pile is successfully taken to plunging failure.

4.2 GRADE BEAMS AND PILE CAPS

A crushable void filler material should be placed beneath all grade beams, and any pile cap that extends beyond the perimeter of an underlying pile, to accommodate soil expansion due to frost action or seasonal soil moisture variations. The void filler should be non-degradable and pest

resistant with no pest nutrient value (Beaver Plastics "Frost Cushion", or equivalent). The appropriate thickness of the void filler should be determined by the manufacturer. The grade beams and pile caps should be designed to withstand upward forces equivalent to the crushing strength of the void filler material.

An adfreeze bond-breaker must be applied to the sides of grade beams and pile caps in areas of the proposed development that will be exposed to frost conditions (i.e. within the depth of frost penetration) at any time during or after construction.

Water ponding within the void filler under grade beams and pile caps could result in frost jacking of the grade beam or pile caps in areas exposed to freezing. Fill placed against the interior face of grade beams and against pile caps should thus be comprised of low to medium plastic inorganic clay or clay till. The clay or clay till should be placed in lifts with compacted thickness of 150mm, at a minimum of 98% of standard proctor maximum dry density at or slightly above optimum moisture content.

4.3 SUBSURFACE UTILITIES

Subsurface utilities referred to in this section include underground water and sewer services. It is assumed that these utilities will be installed at depths ranging from approximately 2m to 5m below final subgrade elevation. Franchise utilities (telephone, cable, natural gas, electricity, etc.) are typically installed in shallow trenches in accordance with the requirements of the individual utility owner.

Installation of subsurface utilities, pipe bedding and backfill should be undertaken in accordance with the requirements of the local municipal authority.

4.3.1 Trench Excavations

Trench excavations must conform to the guidelines in the Alberta Occupational Health and Safety Act. Temporary trench side slopes (open for less than 7 days) of 1 vertical to 1 horizontal may be used to a depth of 1.5m above the base of the excavation. Shallower side slopes may be required if significant groundwater seepage is encountered or if the trench advances through wet soils or sand. Steeper excavations may be considered if engineered safety cages or shored/braced construction is used. All trenches should be monitored regularly for seepage and sloughing, especially after periods of precipitation.

Surcharges, such as material excavated from the trench, should be placed/stockpiled at least 3.0m (or one trench depth, whichever is greater) away from the top edge of the trench. Vehicle traffic should not be allowed within 1.0m of the top edge of the trench.

Every effort should be made to ensure trenches are excavated and backfilled on the same day (at least to a height sufficient to resist buoyant uplift).

Water accumulation was recorded on completion of drilling at depths ranging from 6.70m to 9.75m below grade. Seepage was noted at depths as shallow as 4.50m from grade. The ingress of groundwater into trench excavations is not anticipated in significant amounts and should not have an adverse effect on the installation of subsurface utilities. Perched groundwater may be encountered if the trench excavation intercepts an occasional discontinuous seam of water-bearing silt or sand. Normal pumping and dewatering techniques should be adequate to control any ingress of groundwater. Groundwater should not be allowed to pond on the base of the excavation. Surface water flow should be directed away from trenches and must not be allowed to pond near the edge of the excavation.

The base of the trench excavations will likely be founded in either native inorganic clay, sand or silt. Trench bases founded on the clay should be relatively stable. Trench bases founded on silt or sand may be easily disturbed. If the trench base softens as a result of groundwater infiltration or heavy rainfall, or becomes disturbed during excavation, stabilization of the base may be required before placing the pipe. Stabilization of the trench base should consist of over excavating to a depth of 600mm, placing a woven Class 2 geotextile on the subcut base, and backfilling to the original base elevation with washed rock. The washed rock should be wrapped in the geotextile to provide a base for placement of the pipe bedding.

4.3.2 Pipe Bedding

The type and placement requirements for pipe bedding material should adhere to the specifications of the local municipal authority. In the absence of placement requirements, pipe bedding material should be compacted to a minimum of 95% of Standard Proctor maximum dry density at or slightly below (0% to 2%) its optimum moisture content.

4.3.3 Backfill and Compaction

Native inorganic material excavated from a trench may be reused to backfill the trench provided the required degree of compaction can be obtained. These materials should be replaced in the same vertical sequence as they were encountered while in place. Moisture conditioning of excavated soils may be required, before reuse, in order to achieve the required compaction. Topsoil and organic clay are not acceptable as trench backfill material.

Trench backfill placement requirements issued by the local municipal authority should be followed. In the absence of local specifications, utility trench backfill should be comprised of native material placed in 150mm lifts compacted to 95% of Standard Proctor maximum dry density from the pipe zone to within 1.5m of the final subgrade elevation, and 98% of Standard

Proctor maximum dry density within the upper 1.5m. The uppermost 150mm of the subgrade (final lift) should be compacted to 100% of Standard Proctor maximum dry density at or slightly above the optimum moisture content. All compaction must be verified by density testing.

4.3.4 Frost Protection of Shallow Utilities

Non-insulated buried water lines, sewer lines containing water or fire water lines must have a minimum frost cover of 3.3m below final grade in areas where granular fill is used to backfill the trench. For trenches backfilled using cohesive fill, the depth of frost cover should be at least 2.7m below final grade.

Pipes buried with less than the recommended soil cover must be protected with insulation to prevent frost effects. High strength extruded polystyrene (e.g., Dow Highload 40 or equivalent) could be considered as an insulation option. The local municipal authority's insulation requirements should be followed. In the absence of local specification, the insulation manufacturer's recommendations for placement and thickness of insulating material for protection of shallow utilities should be followed.

For trenches underlying roadways, the insulation should have a minimum depth of cover of 600mm. A 100mm thick sand layer is recommended directly over and underlying the insulation for cushioning. In areas with insulation, the thickness of soil cover overlying the top of pipe zone backfill should be at least 1.0m.

For other buried utilities which are not sensitive to frost-related movement, such as electrical conduits, there are no depth restrictions due to frost. However, such utilities placed within the frost zone must be capable of accommodating seasonal movements on the order of 50mm. If these utilities are not capable of accommodating movements due to frost, they should either be installed below the frost penetration depth or insulated.

As an alternative to extruded polystyrene, cellular concrete may be considered for frost protection. In a trench application, the final design of the cellular concrete should be carried out by the supplier as it is a function of surface cover and other factors.

4.4 UNDERGROUND PARKADE

The following recommendations are based on the assumption that the base of the underground parkade excavations will be established at a depth of approximately 3.0m below site current grades. Should this not be the case, Shelby should be notified to assess the effect, if any, of grade changes on our recommendations.

4.4.1 Excavation/Construction

The groundwater elevation measured in the standpipes indicates that the groundwater table at the site is at a depth of approximately 7.5m below grade, approximately 4.5m below the anticipated depth of excavation. However, based on wet subsurface soil conditions, groundwater seepage from perched groundwater is expected at shallower depths in some areas of the site. Considering the above, ingress of groundwater into the parkade excavation during construction may be encountered in some areas and as such de-watering equipment should be available on site prior to excavation and utilized as required to maintain a dry excavation.

The soil encountered at the base of the excavations is generally expected to consist of low to medium plastic native clay/silt or silty sand. The native clay encountered at the base of the excavation is generally expected to provide a reasonable working surface. The sand/silt, if encountered, may be sensitive to disturbance. The base of the excavation may become disturbed during the normal course of construction and soften during periods of precipitation. Care should be taken during excavation and construction of the parkade to minimize disturbance of the base of the excavation. A mud-slab may be considered to protect the subgrade during construction.

Any abandoned foundation element encountered during subgrade preparation (such as piles, pile caps, grade beams or foundation walls associated with the former/to be demolished onsite structures) should be removed to at least 600mm below the final subgrade elevation to avoid hard spots which could reflect through the new floor slab. Excavations for removal of these elements should be sloped back to avoid an abrupt change in subgrade support and backfilled in 150mm thick lifts with well-graded granular material such as 20mm crushed gravel. The granular material should be compacted to 98% of Standard Proctor maximum dry density at or slightly below (0% to 2%) its optimum moisture content.

If the sides of the excavation cannot be sloped back to comply with Occupational Health and Safety (OH&S) regulations, then the excavation will have to be shored to prevent collapse of adjacent properties/roadways. Recommendations for temporary shoring are provided in the following section.

Concrete should be placed using a pump truck operating remotely from the excavation. Construction traffic should be kept to a minimum.

The subgrade soils should not be allowed to desiccate or be allowed access to free water during construction. If allowed access to water, or allowed to desiccate and then regain moisture the clay will develop swelling pressures with the resultant potential for damage to facets of the proposed development that are in contact with the swelling soils.

4.4.2 Temporary Shoring

Temporary shoring, if required, should be designed by a qualified structural engineer.

The soils encountered during excavation will generally comprise surficial fill underlain by clay, silt or sand.

It is expected that a flexible cantilevered (non-restrained) shoring wall will be feasible for use for this project. Earth pressures behind a cantilevered shoring wall may be calculated using the figure provided on Figure 9, Appendix II, and the parameters in Table 4.3 below.

Table 4.3: Soil Parameters for Shoring Design

Soil Type	Internal Friction Angle	Unit Weight (kN/m ³)	Earth Pressure Coefficient		
			Active (k _a)	Passive (k _p)	At Rest (k _o)
Fill/Clay	20°	19	0.49	2.04	0.66
Silt/Sand	30°	21	0.33	3.03	0.50

The use of k_a , k_o , k_p or a combination thereof to design the wall is a function of the allowable wall movement, i.e.: conditions that exist behind the top of the wall. Shoring designed on k_a is expected to rotate approximately 1% to 2% of the height of the shoring resulting in some loss of grade behind the wall. We have assumed the shoring will be designed as flexible, yielding wall allowed to rotate about some point beneath its base and we have additionally assumed the that wall will be drained so as to preclude the build-up of hydrostatic pressure behind the wall.

In the case of a soldier pile and lagging shoring wall, passive resistance (below the base of the excavation) should be considered to apply over an area equal to three times the pile diameter.

Lateral earth pressures for soils located below the water table should be calculated using the effective unit weight of the soil. That is, the unit weight of water should be subtracted from the value in the table above for soils below the groundwater table.

It must be noted that the forgoing soil parameters are indicative of soils encountered on site at the test hole locations only and are not necessarily representative of the soils that would be retained (i.e., soils between test hole locations or located off site). Additionally, the forgoing soil parameters do not include a safety factor.

4.4.3 Wall Pressures

The parkade walls should be designed to resist the pressure of backfill soil placed adjacent to the wall. The following equation may be used to estimate the magnitude of this pressure.

$$P = 9.4D + 0.5S$$

Where: P = Pressure (kPa)
D = Depth below exterior grade (m)
S = Surcharge, if any, adjacent to the wall (kPa)

This equation does not account for hydrostatic pressures, and consequently a foundation drainage system must be provided at the base of the wall as described in Section 4.4.4 below. In addition, this equation is only valid for the backfill materials and placement recommendations indicated below.

Backfill placed against the walls should be comprised of clean sand (or other suitably free draining granular material), to within 300mm of the surface. A suitable synthetic drain connected directly to the foundation wall could be used in place of granular backfill material. The upper 300mm of backfill should be comprised of a clay cap. Backfill should be modestly compacted, in 200mm thick lifts, to 93% of Standard Proctor maximum dry density. Over-compaction should be avoided to prevent excessive pressures that may damage the wall.

It must be recognized that due to the relatively low degree of compaction of backfill, subsidence of the backfill will occur with associated deflection and possible cracking of hard landscaping features such as sidewalks and pavements. Additionally, subsidence of backfill may result in loss of grade adjacent to the structure. The use of granular backfill will reduce potential backfill consolidation.

4.4.4 Foundation Drainage System

A foundation drainage system must be installed to prevent any unexpected ingress of groundwater into the parkade and elevator pit.

The foundation drainage system should be comprised of weeping tile installed around the entire perimeter of the foundation. As perched groundwater is expected to be present in some areas of the site, an underslab drainage system beneath the parkade floor slab is also required. The collection and discharge of the underslab drainage system should be separate from that for the perimeter weeping tile system. Additionally, the collection and discharge of the perimeter weeping tile system for the elevator pit should be separate.

General conceptual recommendations for the drainage system are provided below. Detailed design of the drainage system should be undertaken by a qualified mechanical engineer.

The underslab drainage system should be comprised of a series of trench drains installed beneath the floor slab. The drains could be comprised of perforated weeping tile (minimum 100mm diameter) bedded in washed gravel, with the weeping tile and gravel wrapped within geotextile filter fabric, or with suitable synthetic trench drains. To interconnect the drains, they should be overlain by a drainage layer comprised of free draining granular material or an acceptable synthetic sheet drain. Any use of synthetic drains should follow the manufacturer's recommendations with respect to installation and interconnections.

If a mud slab is used to protect the subgrade, the underslab drainage system should be placed overlying the mud slab. A non-woven geotextile should be placed between the drainage layer and the overlying granular slab support materials to provide separation. If a mud slab is not used, the drainage layer should be placed overlying the subgrade with the trenches subcut into the subgrade. The entire drainage system should be wrapped with a non-woven geotextile. The subgrade between drains should be sloped to allow for lateral drainage downward to the adjacent drains.

The underslab drains should be installed around the entire perimeter of the parkade (approximately 1.0m inside the foundation wall) with interior laterals spaced no more than approximately 6m to 8m apart. The underslab drains should be installed as soon as the excavation is complete, and connected to a temporary sump pit to control any ingress of groundwater and protect the subgrade during construction. Care must be exercised during construction to prevent the drains from becoming damaged.

The perimeter weeping tile should consist of perforated weeping tile drainage pipe (minimum 100mm minimum diameter) or suitable synthetic drain. Weeping tile, if used, must be bedded in clean washed gravel with the weeping tile and granular materials wrapped in a geotextile filter fabric. The perimeter drainage system should be installed at the same as the elevation as the base of the underslab trench drains, approximately 500mm to 600mm below the top of the parkade slab, and should be connected to the wall drainage materials (as discussed in Section 4.4.3 above). The perimeter drainage system for the elevator pit should be approximately 500mm to 600mm below the top of the elevator pad slab.

The foundation drainage system must be supplied with a positive source of drainage. If possible, the drainage system should be connected directly to the municipal storm sewer system with a gravity pipe. Alternatively, sump pits and pumps should be installed around the perimeter of the parkade and connected to the foundation drainage system. Sump pits and pumps must be maintained on a regular basis.

Consolidation within the backfill zone adjacent to the parkade is expected resulting in a loss of grade adjacent to the building. Grades adjacent to the building should be inspected on a regular basis and any grade loss corrected as required to maintain proper drainage.

4.4.5 Parkade Floor Slab Support

A grade-supported parkade floor slab should be underlain directly by a code-compliant air barrier and should remain structurally separate from foundation components.

Subsequent to excavation to design subgrade elevation, the exposed subgrade would generally comprise firm clay/silt or very loose sand and should be inspected by qualified geotechnical personnel.

Any abandoned foundation element encountered during subgrade preparation (such as piles, pile caps, grade beams or foundation walls associated with the former onsite structures) should be removed to at least 600mm below the final subgrade elevation to avoid hard spots which could reflect through the new floor slab. Excavations for removal of these elements should be sloped back to avoid an abrupt change in subgrade support and backfilled in 150mm thick lifts with well-graded granular material such as 20mm crushed gravel. The granular material should be compacted to 98% of Standard Proctor maximum dry density at or slightly below (0% to 2%) its optimum moisture content.

A minimum 150mm thick capillary break comprised of well-graded gravel (20mm maximum sized, with maximum 10% passing the 0.08mm sieve) should be placed between the air barrier and the subgrade. The gravel should be compacted to 100% of Standard Proctor maximum dry density. Alternate means of providing for a capillary break may be considered.

The Alberta Building Code (the Code) requires that occupied spaces in building with residential occupancies be protected from the ingress of soil gas and radon. A parkade that is fully isolated from the occupied areas with a code compliant Air Barrier and adequate venting and air exchange may act as an effective depressurization layer for the purposes of the code. Notwithstanding the above, the Mechanical Consultant (responsible for the installation of HVAC system for this project) should be contacted to determine acceptable methods suitable to the jurisdiction. If a granular layer rough-in for subfloor depressurization is required, Shelby should be contacted to provide additional recommendations related to the rough-in and revised recommendations for floor slab support.

4.4.6 Parkade Ramp Support

A grade supported concrete ramp may be used provided that the base of the excavation is founded in native soils or new engineered fill. The excavation should be inspected prior to casting concrete to verify that the base of the excavation is founded in competent soils.

Any fill and/or organic material should be removed from the footprint of the ramp. Subsequent to any additional excavation required to achieve the design subgrade elevation, the upper 150mm of the exposed subgrade should be scarified and compacted to 98% of the Standard Proctor maximum dry density at or slightly over optimum moisture content.

Any abandoned foundation element encountered during subgrade preparation (such as piles, pile caps, grade beams or foundation walls associated with the former onsite structures) should be removed to at least 600mm below the final subgrade elevation to avoid hard spots which could reflect through the new ramp. Excavations for removal of these elements should be sloped back to avoid an abrupt change in subgrade support and backfilled in 150mm thick lifts with well-graded granular material such as 20mm crushed gravel. The granular material should be compacted to 98% of Standard Proctor maximum dry density at or slightly below (0% to 2%) its optimum moisture content.

Imported fill material required to raise the grade of the site to design subgrade elevation, should be comprised of low to medium plastic clay or clay till. The material should be compacted in 150mm thick lifts to a minimum of 98% of the Standard Proctor maximum dry density at or slightly above its optimum moisture content. The final lift (upper 150mm) of the subgrade should be compacted to 100% of Standard Proctor maximum dry density at or slightly above the optimum moisture content.

The subgrade should not be permitted to dry out or desiccate during construction. The subgrade beneath the ramp should not be permitted access to free water during the service life of the ramp. The ramp should be constructed with a reverse crown (i.e., concave) to prevent surface water from draining towards the sides of the ramp where it may access the subgrade through the gap that commonly exists between a grade supported ramp and the retaining walls.

A grade supported parkade ramp should be supported by 150mm of crushed gravel underlying a minimum of 50mm of compacted leveling sand. A drainage system must be provided at the base of the ramp designed to prevent the build-up of hydrostatic pressure beneath the ramp and to provide positive drainage for surface water.

A grade supported ramp must remain structurally separate from the retaining walls and the existing foundation components. Insulation and heating may be required to maintain the ramp snow free and to prevent excessive frost heave.

Free edges of concrete pavement subjected to vehicle traffic should have the concrete thickness increased by 20% over a linear distance of 1.5m.

4.5 BUILDING PAD GRADING

The subgrade will be comprised predominately of cohesive (clay-based) material, which is subject to volumetric changes with a change in moisture content. To mitigate the possibility of swelling (upon wetting) or shrinkage (upon drying) of the cohesive soils (and resulting vertical movements of overlying surfacing such as floor slabs or pavements), care must be taken during and after construction to keep changes in insitu moisture content to a minimum.

Surface water cannot be permitted to “pond” or “pool” adjacent to the buildings and infiltrate the subgrade adjacent to or beneath the buildings. Water, if allowed to “pond” or “pool” adjacent to the buildings, could infiltrate the clay subgrade beneath the buildings causing the clay to swell and the floor slab to heave. Water ponding within the void filler under grade beams can result in frost jacking of grade beams.

Water discharged at grade from downspouts or sump pumps must discharge well away from the buildings or be connected directly into the storm sewer system. Concrete splash pads are recommended if water is discharged to the surface. Neither surface water nor discharge from downspouts or sump pumps should be introduced into, or allowed to recycle back into, the foundation drainage systems. A foundation drainage system should not be used to collect and control surface water from any source. Ponding and water Infiltration can be mitigated by placing an upper clay cap (minimum 600mm thick) adjacent to the grade beams/foundation walls, extending a minimum of 3m outward from the perimeter of the structure.

Site grading adjacent to the buildings must be designed so as to direct all surface runoff (water), from any source, away from the building. A minimum grade of 2% away from the buildings is recommended for hard landscaped areas (i.e.: concrete or asphalt), while a minimum grade in the range of 10% is recommended for soft landscaped areas. These grades should be maintained for a distance of at least 3m from the buildings for the lifetime of the structures.

Backfill consolidation adjacent to foundation walls or grade beams will most likely occur over time and may be sufficient to cause a loss of grade in soft landscaped areas and possibly grade reversal in hard landscaped areas. The owner should periodically inspect all grades adjacent to the buildings and correct any grade loss as required to maintain good positive surface drainage away from the buildings. Missing or damaged downspout extensions or splash pads should be repaired or replaced immediately.

4.6 CONCRETE

No significant concentrations of soluble sulphates were measured in tested soil samples recovered from this site. Type GU, GUb or equivalent Portland cement may be used for production of concrete in contact with the existing onsite soils.

Concrete having a minimum 28-day compressive strength of 25 MPa is acceptable for foundation concrete. Concrete exposed to freeze-thaw cycles and/or de-icing chemicals may have different strength requirements as well as air entrainment and water-to-cementitious-materials ratio requirements. Shelby may be able to provide further direction upon request.

Any imported fill that will be in contact with concrete should be tested for soluble sulphates before use, and the above recommendations for cement type re-evaluated.

4.7 SEISMIC SITE CLASSIFICATION

Based on the results of the field investigation, combined with our experience in the area and a review of published geological information pertaining to the region, the Seismic Site Classification for the site shall be taken as "C".

4.8 ASPHALT PAVEMENTS

The following recommendations assume that the final grade of asphalt pavements will be approximately coincident with surface grades as existed at the time of this investigation.

Frozen materials must not be used for fill. Construction of pavement structures (including base support) must not be undertaken over frozen subgrade soils.

The recommendations below are not applicable to areas outside of the site/study area. The authority having jurisdiction should be consulted with respect to offsite access improvements such as crossings or turning lanes.

4.8.1 Subgrade Preparation

Pavement areas should be inspected and stripped of any surficial topsoil, organic or unsuitable material.

After any additional stripping (if required to reach the final subgrade elevation) the exposed subgrade should be inspected by qualified geotechnical personnel and then scarified to a depth of 150mm and compacted to a minimum of 98% of Standard Proctor maximum dry density at or slightly over optimum moisture content. Considering the potential presence of fill soils with significant organics or silt content in some areas of the site, additional subgrade preparation

measures such as the use of a geogrid, high modulus geotextile or cement stabilization (or a combination thereof) may be required based on the exposed subgrade conditions at the time of construction”.

Fill material used to raise the grade of pavement areas should be comprised of low to medium plastic clay or clay till. The material should be compacted in 150mm thick lifts to a minimum of 98% of the Standard Proctor maximum dry density at or slightly above its optimum moisture content. The final lift (upper 150mm) of the subgrade should be compacted to 100% of Standard Proctor maximum dry density at or slightly above the optimum moisture content. The upper surface of the subgrade should be shaped to parallel the final grade of the overlying asphalt pavement.

Prior to placement of granular base course material, the prepared subgrade should be proof rolled using a heavily loaded gravel truck or equivalent piece of equipment to identify any soft areas. Proof rolling should be completed under the supervision of qualified Geotechnical personnel. Recommendations pertaining to the repair of soft areas can only be made at the time of inspection.

4.8.2 Structure Design

The light-duty traffic design below is for areas subjected to passenger vehicle traffic only, such as parking stalls. The heavy-duty design below is for areas subjected to occasional truck traffic such as entrances and drive aisles (equivalent to 2 typical semi-tractor units with 5 axles per day). Either structure is capable of supporting emergency response traffic (including standard fire-fighting equipment). If traffic volumes and/or vehicle types different than the above assumptions are expected, Shelby should be contacted to provide alternate structure recommendations.

A California Bearing Ratio (CBR) of 2.0 has been estimated for the subgrade if prepared as outlined above. Based on this, the following asphalt pavement structures are recommended:

Table 4.4: Asphalt Pavement Structures

Lift	Light Duty (parking stalls)	Heavy Duty (drive aisles)
Asphalt	75mm	100mm
Granular Base (20mm crushed gravel)	250mm	300mm

The granular base material should meet the requirements for City of Edmonton Designation 3 Class 20 material, and should be compacted to 100% of the Standard Proctor maximum dry density in 150mm lifts.

The asphalt concrete should meet the requirements for City of Edmonton Type "10mm-HT", compacted to a minimum of 94% of the maximum relative density. Alternatively, the asphalt concrete could meet the requirements for City of Edmonton mix type "ACO", if available. This asphalt type should be compacted to 97% of Marshall density.

To ensure a high level of performance from pavement sections, the subgrade must not be allowed to dry and/or become wetted prior to or subsequent to construction.

4.8.3 Drainage

Water cannot be permitted to pond or pool on the surface of the asphalt pavement. All surface runoff must be conveyed to a perimeter drainage ditch or to catch basins, to prevent water infiltration into the subgrade. Asphalt pavement areas must have a minimum grade of 1%.

Catch basin barrels should be perforated at the interface of the granular base and underlying prepared subgrade. Some method of preventing ingress of soil into the sewer system must be implemented.

4.8.4 Maintenance

A regular scheduled maintenance program will extend the life of the asphalt pavement. Asphalt pavements should be inspected on an annual basis and any cracks sealed to prevent water from infiltrating the subgrade.

5.0 SUPPLEMENTARY RECOMMENDATIONS FOR PARCEL A

With reference to Parcel A, Shelby has reviewed the existing 2016 Geotechnical report prepared by Thurber. Based on the review, Shelby is of the opinion that the report is generally valid for the 15-storey building. It should however be noted that due to anticipated higher column loads, footings may not be feasible for the proposed development except that radon and pavement recommendations were not included in the report. Shelby can confirm feasibility of footings once design column loads and preliminary footing sizes are known.

Radon and pavement recommendations presented in Sections 4.4.5 and 4.8 of this report are also applicable to the proposed building in Parcel A.

6.0 CONSTRUCTION TESTING AND MONITORING

Appropriate laboratory and field testing and monitoring by qualified geotechnical personnel is recommended during any earthworks (i.e., site grading, excavating, backfilling, building/pavement subgrade preparation) and placement of all engineered fill and base or subbase materials, to ensure that suitable site conditions are prepared and that materials consistent with the recommendation herein are used.

The Alberta Building Code specifies full time continuous field review, by a suitably qualified individual, during installation of all deep foundation elements.

7.0 CLOSURE

All services provided by Shelby Engineering Ltd. are subject to our Standard Terms and Conditions, which are attached in Appendix I.

Respectfully Submitted,
SHELBY ENGINEERING LTD.



July 16/2021
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PERMIT TO PRACTICE SHELBY ENGINEERING LTD	
RM SIGNATURE:	
RM APEGA ID #:	94608
DATE:	July 16, 2021
PERMIT NUMBER: P003580	
The Association of Professional Engineers and Geoscientists of Alberta (APEGA)	

APPENDIX I

Standard Terms and Conditions

**STANDARD TERMS AND CONDITIONS FOR THE PROVISION OF SERVICES
BY SHELBY ENGINEERING LTD.**

1. "The services ("the Services") performed for the client (the "Client") by Shelby Engineering Ltd. ("Shelby") described in the report to which these Standard Terms and Conditions are attached (the "Report") have been conducted in a manner consistent with the level of skill ordinarily exercised by members of the engineering profession currently practicing in the jurisdiction in which the Services have been provided."
2. In consideration of the provision of the Services, the Client agrees to the limitation of liability provisions herein contained, both on its own behalf, and as agent on behalf of its employees and principals.
3. The total amount of all claims the Client may have against Shelby with respect to the Services, including, without limitation, claims in tort or contract, shall be strictly limited to the amount of the fee charged to the Client by Shelby for the Services. Shelby shall not be liable for loss, injury or damage caused by delays beyond Shelby's control, or for any indirect, economic or consequential loss, injury or damage incurred by the Client, including, without limitation, claims for loss of profits, loss of contracts, loss of use, loss of production or business opportunity, loss of contracts or continued overhead expense. No claim shall be brought by the Client against Shelby more than two (2) years after completion of the Services or termination of the agreement to provide the Services.
4. The Client shall have no right to set off against any amounts owed to Shelby with respect to the Services.
5. The Client agrees that Shelby's employees and principals shall have no personal liability with respect to the Services and the Client shall make no claim or bring any proceedings of any kind whatsoever whether in contract, tort or any other cause of action in law or equity, against Shelby's employees and principals in their personal capacity.
6. The Client acknowledges that the Services entail an investigation which by its nature involves the risk that certain conditions between points investigated will not be detected, and that certain other conditions may change with time after provision of the written report of the Services. The Client acknowledges and accepts such risk and is aware that the Report can only provide for the conditions at the investigated points at the time of investigation. Extrapolation between the investigated points is at the Client's risk. If the Client requires additional or special investigations outside the scope of the Report, the Client must request such additional investigations from Shelby.
7. The Report has been prepared for a specific site and in light of the specific purposes communicated to Shelby by the Client. Shelby accepts no responsibility for the findings contained in the Report if applied to a different site, or if there is a material change in the purposes communicated to Shelby by the Client. The information and opinions described in the Report are provided solely for the benefit of the Client. NO OTHER PARTY MAY USE OR RELY UPON THE REPORT OR ANY PORTION THEREOF WITHOUT THE WRITTEN CONSENT OF SHELBY. The Client shall maintain confidentiality of the Report and ensure that the Report is not distributed to third parties. The Client hereby agrees to indemnify Shelby for any claims brought against Shelby by third parties and arising out of the Client's failure to maintain the confidentiality required under this paragraph.
8. Except as stipulated in the Report, Shelby has not been retained to address, investigate or consider, and has not addressed, investigated or considered environmental or regulatory issues with respect to the site on which the Services have been performed. Notwithstanding the foregoing, Shelby may be required to disclose to regulatory bodies certain hazardous conditions discovered through provision of the Services, and the Client shall not make any claim against Shelby for such disclosure.

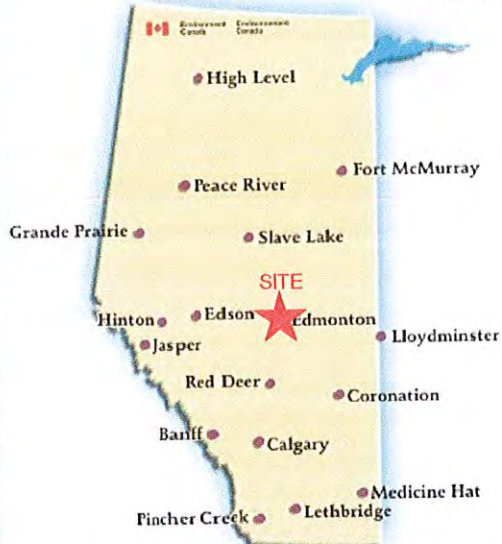
July 2005Revised



APPENDIX II

Figures

ALBERTA



NOTES:



MAPS OBTAINED FROM GOVERNMENT OF CANADA WEBSITE
AND MICROSOFT CORPORATION BING MAPS

CLIENT:

FIVE OAKS INC.

PROJECT:

ROSSDALE CENTRE MULTI-FAMILY
DEVELOPMENT
9803, 9806-9842 100 ST. NW
EDMONTON, AB



SITE LOCATION PLAN

OWN BY:	CKD BY:	REVISION:
TD	HKC	1
PROJECT NO.:	DATE:	FIGURE NO.:
1-22776	JUNE 2021	1



NOTES:



SCALE: 1:1500

TBM, TOP OF CONCRETE SLAB AT BREWERY DOOR
ASSUMED ELEVATION: (100.00m)

CLIENT:

FIVE OAKS INC.

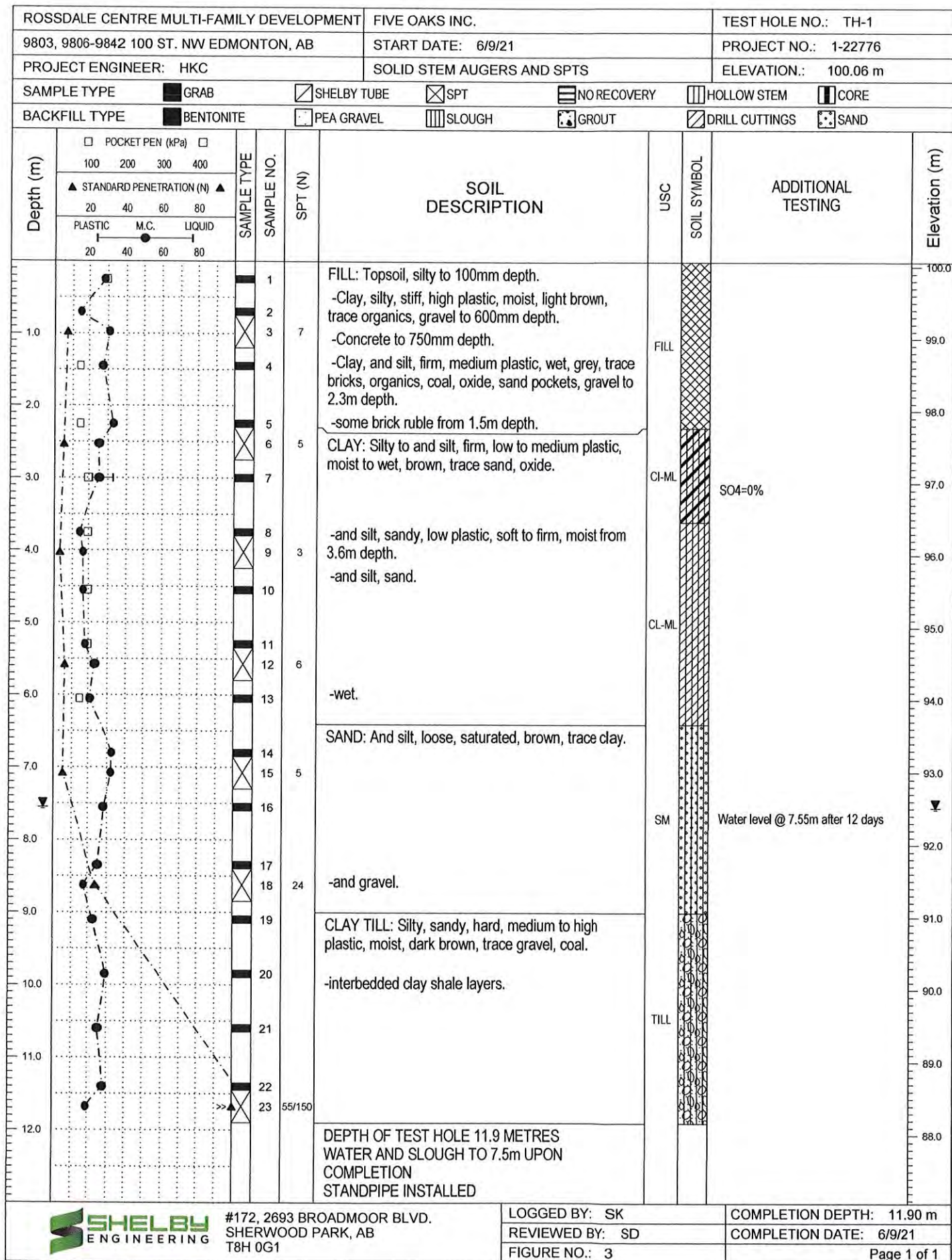
PROJECT:

ROSSDALE CENTRE MULTIFAMILY
DEVELOPMENT
9803, 9806-9842 100 ST. NW
EDMONTON, AB



SITE PLAN

OWN BY:	CKD BY:	REVISION:
TD	HKC	1
PROJECT NO.: 1-22776	DATE: JUNE 2021	FIGURE NO.: 2



ROSSDALE CENTRE MULTI-FAMILY DEVELOPMENT		FIVE OAKS INC.		TEST HOLE NO.: TH-2	
9803, 9806-9842 100 ST. NW EDMONTON, AB		START DATE: 6/9/21		PROJECT NO.: 1-22776	
PROJECT ENGINEER: HKC		SOLID STEM AUGERS AND SPTS		ELEVATION.: 100.65 m	
SAMPLE TYPE ☒ GRAB		☒ SHELBY TUBE	☒ SPT	☐ NO RECOVERY	☐ HOLLOW STEM
BACKFILL TYPE ☒ BENTONITE		☐ PEA GRAVEL	☐ SLOUGH	☐ GROUT	☒ DRILL CUTTINGS
□ POCKET PEN (kPa) □ 100 200 300 400 ▲ STANDARD PENETRATION (N) ▲ 20 40 60 80 PLASTIC M.C. LIQUID 20 40 60 80 SOIL DESCRIPTION USC SOIL SYMBOL ADDITIONAL TESTING Elevation (m)					
Depth (m) 1.0 2.0 3.0 4.0 5.0 6.0 7.0 8.0 9.0 10.0 11.0 12.0		SAMPLE TYPE SAMPLE NO. SPT (N) 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20 21 22 23 24	FILL: Topsoil, silty, black to 450mm depth. -Clay, silty, firm, medium to high plastic, damp to moist, medium to dark brown, some organics to 1.2m depth. -and silt, low to medium plastic from 750mm depth. CLAY: And silt, firm, low plastic, damp, brown, trace rootlets. SAND: Silty, loose, fine grained, damp to moist, brown, trace clay. -moist. -saturated. -and gravel, some cobbles. SAND TILL: Silty, very stiff, low plastic, moist, light brown, trace clay, gravel, oxide, coal. CLAY TILL: Silty, sandy, hard, medium to high plastic, moist, grey, trace gravel, coal. -interbedded clay shale layers. DEPTH OF TEST HOLE 11.9 METRES WATER AND SLOUGH TO 6.7m UPON COMPLETION BACKFILLED	FILL CL-ML SM TILL TILL	100.0 99.0 98.0 97.0 96.0 95.0 94.0 93.0 92.0 91.0 90.0 89.0 88.0
		#172, 2693 BROADMOOR BLVD. SHERWOOD PARK, AB T8H 0G1		LOGGED BY: SK REVIEWED BY: SD FIGURE NO.: 4	
				COMPLETION DEPTH: 11.90 m COMPLETION DATE: 6/9/21 Page 1 of 1	

ROSSDALE CENTRE MULTI-FAMILY DEVELOPMENT		FIVE OAKS INC.		TEST HOLE NO.: TH-3	
9803, 9806-9842 100 ST. NW EDMONTON, AB		START DATE: 6/9/21		PROJECT NO.: 1-22776	
PROJECT ENGINEER: HKC		SOLID STEM AUGERS AND SPTS		ELEVATION.: 99.84 m	
SAMPLE TYPE ☒ GRAB		☐ SHELBY TUBE		☒ SPT	
☐ NO RECOVERY		☐ HOLLOW STEM		☐ CORE	
BACKFILL TYPE ☒ BENTONITE		☐ PEA GRAVEL		☐ SLOUGH	
☐ GROUT		☒ DRILL CUTTINGS		☐ SAND	

Depth (m)	☐ POCKET PEN (kPa) ☐ 100 200 300 400 ▲ STANDARD PENETRATION (N) ▲ 20 40 60 80 PLASTIC M.C. LIQUID 20 40 60 80	SAMPLE TYPE	SAMPLE NO.	SPT (N)	SOIL DESCRIPTION	USC	SOIL SYMBOL	ADDITIONAL TESTING	Elevation (m)
1.0			1	11	FILL: Clay, silty, stiff, medium plastic, moist, brown and black, some organics, trace to some concrete rubble, brick rubble, wood to 2.2m depth.				99.0
2.0			2						98.0
3.0			3						97.0
4.0			4		-and silt, low plastic from 1.4m depth.				96.0
5.0			5	5	SILT: Firm, low plastic, damp, brown, some sand, trace clay.			SO4=0%	95.0
6.0			6						94.0
7.0			7						93.0
8.0			8		-some sand layers.				92.0
9.0			9	3					91.0
10.0			10		CLAY: And silt, soft to firm, low plastic, saturated, brown, trace oxide.				90.0
11.0			11	5	-and sand.				89.0
12.0			12						88.0
			13		SAND: Silty, loose, saturated, brown, trace clay.				87.0
			14						
			15	42	-gravelly, some cobbles.				
			16		-interbedded clay till layers.				
			17						
			18	38					
			19		CLAY TILL: Silty, sandy, hard, medium to high plastic, damp to moist, dark brown to grey trace gravel, coal.				
			20		-interbedded coal layers from 9.05m to 9.9m depth.				
			21	49					
			22		-interbedded clay shale layers.				
			23						
					DEPTH OF TEST HOLE 11.9 METRES WATER AND SLOUGH TO 6.7m UPON COMPLETION BACKFILLED				

	#172, 2693 BROADMOOR BLVD. SHERWOOD PARK, AB T8H 0G1	LOGGED BY: SK	COMPLETION DEPTH: 11.90 m
		REVIEWED BY: SD	COMPLETION DATE: 6/9/21
		FIGURE NO.: 5	Page 1 of 1

ROSSDALE CENTRE MULTI-FAMILY DEVELOPMENT		FIVE OAKS INC.		TEST HOLE NO.: TH-4	
9803, 9806-9842 100 ST. NW EDMONTON, AB		START DATE: 6/9/21		PROJECT NO.: 1-22776	
PROJECT ENGINEER: HKC		SOLID STEM AUGERS AND SPTS		ELEVATION.: 100.19 m	
SAMPLE TYPE ☒ GRAB		☒ SHELBY TUBE	☒ SPT	☐ NO RECOVERY	☐ HOLLOW STEM
BACKFILL TYPE ☒ BENTONITE		☐ PEA GRAVEL	☐ SLOUGH	☐ GROUT	☐ DRILL CUTTINGS
		☐ SAND			

Depth (m)	POCKET PEN (kPa) 100 200 300 400	STANDARD PENETRATION (N) 20 40 60 80	PLASTIC 20 40 60 80	M.C. 20 40 60 80	LIQUID 20 40 60 80	SAMPLE TYPE	SAMPLE NO.	SPT (N)	SOIL DESCRIPTION	USC	SOIL SYMBOL	ADDITIONAL TESTING	Elevation (m)
1.0							1	7	FILL: Topsoil, black to 75mm depth. -Clay, silty, stiff, medium plastic, moist, brown and grey, some organics, wood to 2.2m depth.	FILL			100.0
2.0							2						99.0
3.0							3	3	SILT: Sandy, very loose, low plastic, damp, brown, trace clay.	SASI			98.0
4.0							4		SAND: And silt, very loose, fine grained, damp to moist, brown, trace coal.	SM			97.0
5.0							5	2					96.0
6.0							6		CLAY: And silt, soft to firm, low plastic, moist, brown, trace to some sand.	CL-ML			95.0
7.0							7	3	-wet to saturated.				94.0
8.0							8						93.0
9.0							9	1	SAND: Silty, very loose, fine grained, saturated, brown.	SM			92.0
10.0							10		SAND TILL: Silty, hard, low to non plastic, moist, brown, some gravel, cobbles, trace clay.	TILL			91.0
11.0							11	61					90.0
12.0							12	37	CLAY TILL: Silty, sandy, hard, medium plastic, moist, dark grey, trace gravel, coal.	TILL			89.0
13.0							13		-interbedded clay shale layers.				88.0
14.0							14	30/50	DEPTH OF TEST HOLE 11.75 METRES WATER AND SLOUGH TO 9.75m UPON COMPLETION BACKFILLED				

	#172, 2693 BROADMOOR BLVD. SHERWOOD PARK, AB T8H 0G1	LOGGED BY: SK	COMPLETION DEPTH: 11.90 m
		REVIEWED BY: SD	COMPLETION DATE: 6/9/21
		FIGURE NO.: 6	Page 1 of 1

ROSSDALE CENTRE MULTI-FAMILY DEVELOPMENT		FIVE OAKS INC.		TEST HOLE NO.: TH-5	
9803, 9806-9842 100 ST. NW EDMONTON, AB		START DATE: 6/9/21		PROJECT NO.: 1-22776	
PROJECT ENGINEER: HKC		SOLID STEM AUGERS AND SPTS		ELEVATION.: 100.28 m	
SAMPLE TYPE ☒ GRAB		☒ SHELBY TUBE	☒ SPT	☐ NO RECOVERY	☐ HOLLOW STEM
BACKFILL TYPE ☒ BENTONITE		☐ PEA GRAVEL	☐ SLOUGH	☐ GROUT	☐ DRILL CUTTINGS
☐ POCKET PEN (kPa) ☐ 100 200 300 400 ☒ STANDARD PENETRATION (N) ☒ 20 40 60 80 PLASTIC M.C. LIQUID 20 40 60 80		SAMPLE NO.		Elevation (m)	
Depth (m)		SPT (N)		SOIL DESCRIPTION	
		1 2 3 4 5 6 7 8 9		100.0 99.0 98.0 97.0 96.0 95.0 94.0 93.0 92.0 91.0 90.0 89.0 88.0	
				USC SOIL SYMBOL FILL SM SO4=0% Test hole dry after 12 days	
		DEPTH OF TEST HOLE 4.25 METRES DRY UPON COMPLETION NO SLOUGH UPON COMPLETION STANDPIPE INSTALLED			

#172, 2693 BROADMOOR BLVD.
SHERWOOD PARK, AB
T8H 0G1

LOGGED BY: SK
REVIEWED BY: SD
FIGURE NO.: 7

COMPLETION DEPTH: 4.25 m
COMPLETION DATE: 6/9/21
Page 1 of 1

ROSSDALE CENTRE MULTI-FAMILY DEVELOPMENT		FIVE OAKS INC.		TEST HOLE NO.: TH-6	
9803, 9806-9842 100 ST. NW EDMONTON, AB		START DATE: 6/9/21		PROJECT NO.: 1-22776	
PROJECT ENGINEER: HKC		SOLID STEM AUGERS AND SPTS		ELEVATION.: 99.90 m	
SAMPLE TYPE ☒ GRAB		☒ SHELBY TUBE	☒ SPT	☐ NO RECOVERY	☐ HOLLOW STEM
BACKFILL TYPE ☒ BENTONITE		☐ PEA GRAVEL	☐ SLOUGH	☐ GROUT	☐ DRILL CUTTINGS
☐ CORE					

Depth (m)	☐ POCKET PEN (kPa) ☐ 100 200 300 400 ▲ STANDARD PENETRATION (N) ▲ 20 40 60 80 PLASTIC M.C. LIQUID 20 40 60 80	SAMPLE TYPE	SAMPLE NO.	SPT (N)	SOIL DESCRIPTION	USC	SOIL SYMBOL	ADDITIONAL TESTING	Elevation (m)
0.0			1		FILL: Topsoil, silty, black to 450mm depth.				99.90
0.5			2		-Clay, silty, stiff, medium to high plastic, moist, brown and black, some organics to 1.35m depth.	FILL			
1.0			3	6	-trace gravel, oxide from 750mm depth.				
1.5			4		CLAY: And silt, firm to stiff, low plastic, wet, brown, trace oxide.				98.0
2.0			5						
2.5			6	9	-trace coal.	CL-ML			97.0
3.0			7						
3.5			8						
4.0			9	4	SILT: Sandy, loose, low to non plastic, moist, brown, trace clay.	SM			96.0
4.25					DEPTH OF TEST HOLE 4.25 METRES DRY UPON COMPLETION NO SLOUGH UPON COMPLETION STANDPIPE INSTALLED			Test hole dry after 12 days	95.0
5.0									94.0
6.0									93.0
7.0									92.0
8.0									91.0
9.0									90.0
10.0									89.0
11.0									88.0
12.0									87.0

	#172, 2693 BROADMOOR BLVD. SHERWOOD PARK, AB T8H 0G1	LOGGED BY: SK	COMPLETION DEPTH: 4.25 m
		REVIEWED BY: SD	COMPLETION DATE: 6/9/21
		FIGURE NO.: 8	Page 1 of 1

SOIL CLASSIFICATION SYSTEM (MODIFIED U.S.C.)

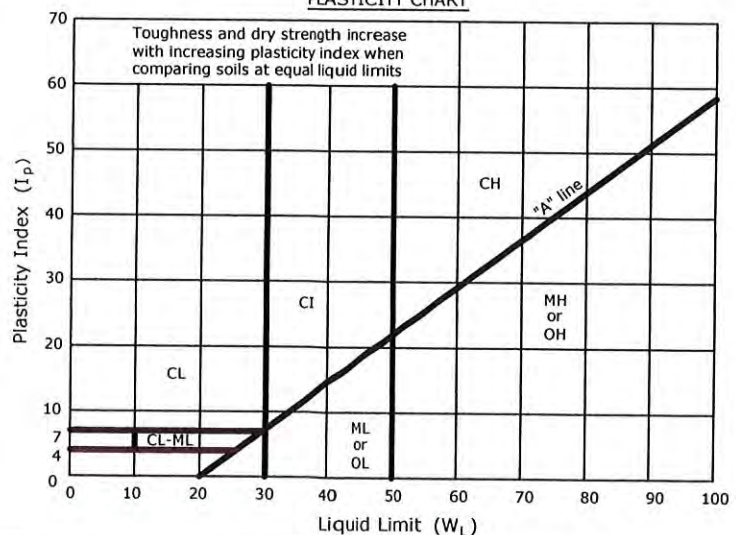
MAJOR DIVISION			GROUP SYMBOL	GRAPHIC SYMBOL	GROUP NAME	LABORATORY CLASSIFICATION CRITERIA	
HIGHLY ORGANIC SOILS			PT		PEAT AND OTHER HIGHLY ORGANIC SOILS	STRONG COLOR OR ODOR, AND OFTEN FIBROUS TEXTURE	
COARSE-GRAINED SOILS MORE THAN 50% RETAINED ON NO.200 SIEVE	GRAVELS MORE THAN 50% OF COARSE FRACTION RETAINED ON NO.4 SIEVE	CLEAN GRAVELS LESS THAN 5% FINES	GW		WELL-GRADED GRAVELS, GRAVEL-SAND MIXTURES, < 5% FINES	$Cu = \frac{D_{60}}{D_{10}} > 4$ $1 \leq Cc = \frac{(D_{30})^2}{D_{10} \times D_{60}} \leq 3$	
			GP		POORLY-GRADED GRAVELS, GRAVEL-SAND MIXTURES, < 5% FINES	NOT MEETING ALL ABOVE REQUIREMENTS	
		DIRTY GRAVELS MORE THAN 12% FINES	GM		SILTY GRAVELS, GRAVEL-SAND-SILT MIXTURES, > 12% FINES	ATTERBERG LIMITS BELOW "A" LINE OR $I_p < 4$	
			GC		CLAYEY GRAVELS, GRAVEL-SAND-CLAY MIXTURES, > 12% FINES	ATTERBERG LIMITS ABOVE "A" LINE OR $I_p > 7$	
	SANDS MORE THAN 50% OF COARSE FRACTION PASSES NO. 4 SIEVE	CLEAN SANDS LESS THAN 5% FINES	SW		WELL-GRADED SANDS, GRAVELLY SANDS, < 5% FINES	$Cu > 6$ and $1 \leq Cc \leq 3$	
			SP		POORLY-GRADED SANDS, OR GRAVELLY SANDS, < 5% FINES	NOT MEETING ALL ABOVE REQUIREMENTS	
		DIRTY SANDS MORE THAN 12% FINES	SM		SILTY SANDS, SAND-SILT MIXTURES, > 12% FINES	ATTERBERG LIMITS BELOW "A" LINE OR $I_p < 4$	
			SC		CLAYEY SANDS, SAND-CLAY MIXTURES, > 12% FINES	ATTERBERG LIMITS ABOVE "A" LINE OR $I_p > 7$	
FINE-GRAINED SOILS MORE THAN 50% PASSES NO. 200 SIEVE	SILTS BELOW "A" LINE ON PLASTICITY CHART; NEGLEGIBLE ORGANIC CONTENT		ML		INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTY SANDS OF SLIGHT PLASTICITY	$W_L < 50$	SEE PLASTICITY CHART BELOW
			MH		INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS, FINE SANDY OR SILTY SOILS	$W_L > 50$	
	CLAYS ABOVE "A" LINE ON PLASTICITY CHART; NEGLEGIBLE ORGANIC CONTENT	CL		INORGANIC CLAYS OF LOW PLASTICITY, GRAVELLY, SANDY, OR SILTY CLAYS, LEAN CLAYS	$W_L < 30$		
		CI		INORGANIC CLAYS OF MEDIUM PLASTICITY, SILTY CLAYS	$30 < W_L < 50$		
		CH		INORGANIC CLAYS OF HIGH PLASTICITY, FAT CLAYS	$W_L > 50$		
	ORGANIC SILTS AND ORGANIC CLAYS BELOW "A" LINE ON PLASTICITY CHART		OL		ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY	$W_L < 50$	
			OH		ORGANIC CLAYS OF HIGH PLASTICITY	$W_L > 50$	

- All sieve sizes mentioned on this chart are U.S. Standard, ASTM E11
- Boundary classifications possessing characteristics of two groups are given combined group symbols. eg. GW-GC is a well-graded gravel-sand mixture with clay binder of between 5% and 12%.
- Soil fractions and limiting textural boundaries are in accordance with the Unified Soil Classification System (ASTM D2487), except that an inorganic clay of medium plasticity (CI) is recognized.
- The following adjectives may be employed to define percentage ranges by weight of minor components (per Canadian Foundation Engineering Manual, 1992):

And - 35% to 50%
(y/ey) - 20% to 35%
Some - 10% to 20%
Trace - 1% to 10%



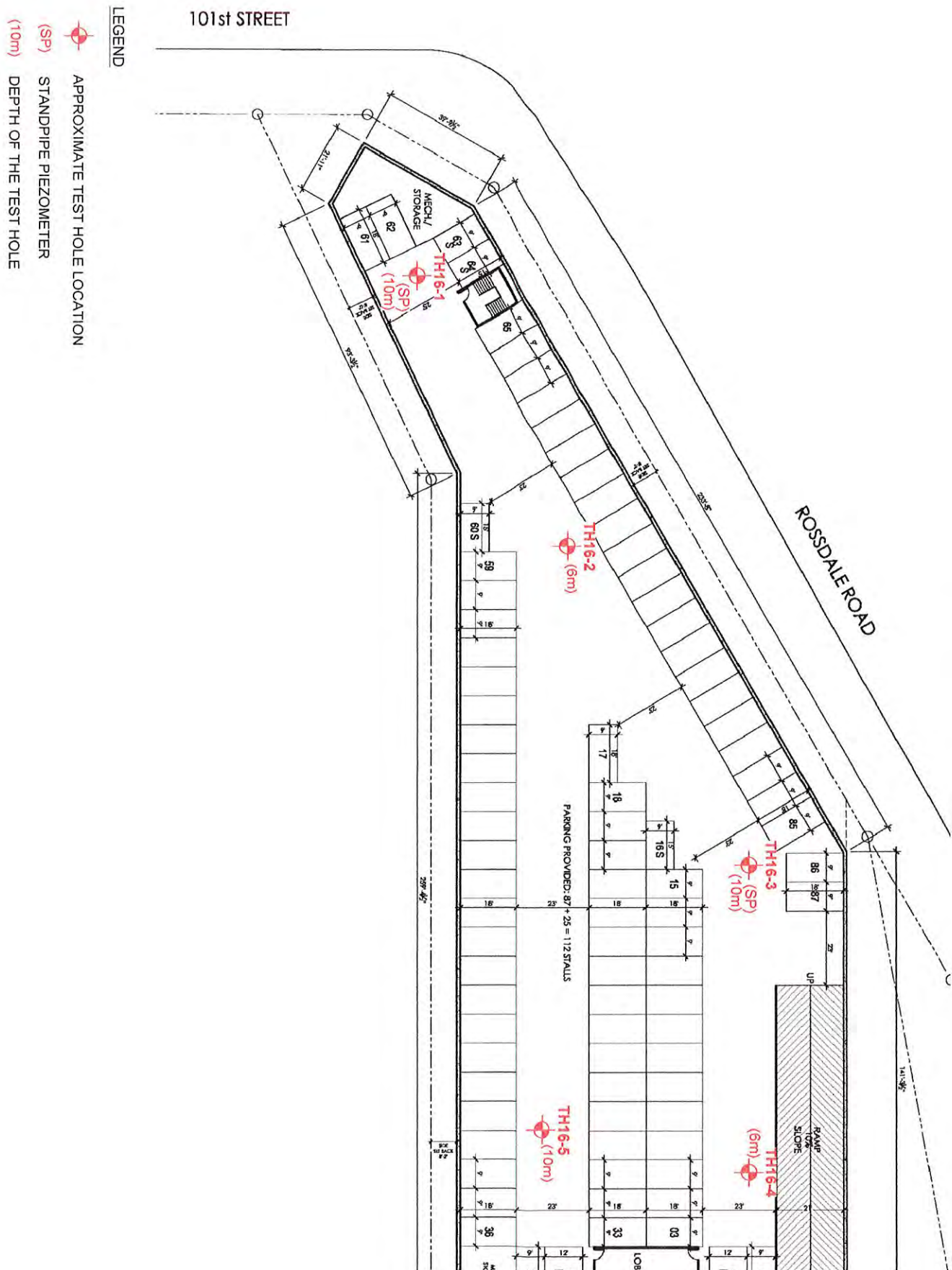
PLASTICITY CHART



SOIL CLASSIFICATION CHART

APPENDIX III

Parcel A Test Hole Logs and Site Plan (From 2016 Thurber Report)





APPENDIX B

Symbols and Terms Used on Test Hole Logs
Modified Unified Soils Classification
Test Hole Logs

SYMBOLS AND TERMS USED ON TEST HOLE LOGS

1. VISUAL TEXTURAL CLASSIFICATION OF MINERAL SOILS

<u>CLASSIFICATION</u>	<u>APPARENT PARTICLE SIZE</u>	<u>VISUAL IDENTIFICATION</u>
Boulders	Greater than 200 mm	Greater than 200 mm
Cobbles	75 mm to 200 mm	75 mm to 200 mm
Gravel	4.75 mm to 75 mm	5 mm to 75 mm
Sand	0.075 mm to 4.75 mm	Visible particles to 5 mm
Silt	0.002 mm to 0.075 mm	Non-Plastic particles, not visible to the naked eye
Clay	Less than 0.002 mm	Plastic particles, not visible to the naked eye

2. TERMS DESCRIBING CONSISTENCY (COHESIVE SOILS ONLY)

<u>DESCRIPTIVE TERM</u>	<u>APPROXIMATE UNDRAINED SHEAR STRENGTH</u>	<u>APPROXIMATE SPT * 'N' VALUE</u>
Very Soft	Less than 10 kPa	Less than 2
Soft	10 - 25 kPa	2 to 4
Firm	25 - 50 kPa	4 to 8
Stiff	50 - 100 kPa	8 to 15
Very Stiff	100 - 200 kPa	15 to 30
Hard	200 - 300 kPa	Greater than 30
Very Hard	Greater than 300 kPa	

} Modified from
National Building
Code

* SPT 'N' Value Standard Penetration Test 'N' Value - refers to the number of blows from a 63.5 kg hammer free falling a height of 0.76m to advance a standard 50mm outside diameter split spoon sampler for 0.3m depth into the undrilled portion of the test hole.

3. TERMS DESCRIBING DENSITY (COHESIONLESS SOILS ONLY)

<u>DESCRIPTIVE TERM</u>	<u>STANDARD PENETRATION TEST (SPT) (Number of Blows per 300 mm)</u>
Very Loose	0 - 4
Loose	4 - 10
Compact	10 - 30
Dense	30 - 50
Very Dense	Over 50

} Modified from
National Building
Code

4. LEGEND FOR TEST HOLE LOGS

SYMBOL FOR SAMPLE TYPE

 Shelby Tube	 A-Casing
 SPT	 Grab
 No Recovery	 Core

SYMBOLS USED FOR TEST HOLE LOGS

●	MC - Moisture Content (% by weight) of soil sample
▼	Water Level
■	SPT Standard Penetration Test 'N' Value (Blows/300mm)
▲	CPen Shear Strength determined by pocket penetrometer
CVane	Shear Strength determined by pocket vane
Cu	Undrained Shear Strength determined by unconfined compression test
SO ₄ %	Percent (%) of water soluble sulphate ions



THURBER ENGINEERING LTD.

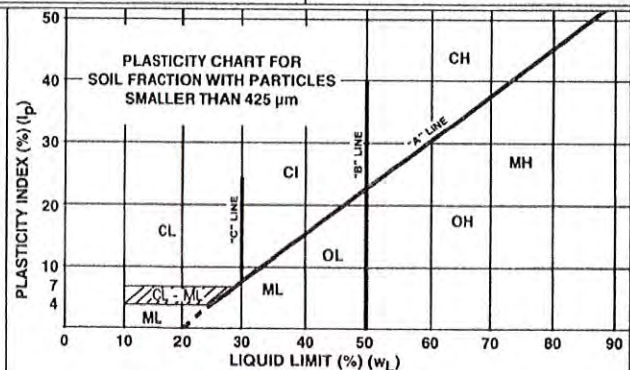
MODIFIED UNIFIED CLASSIFICATION SYSTEM FOR SOILS

(MODIFIED BY PFRA, 1985)

MAJOR DIVISION			GROUP SYMBOL	THURBER LOG SYMBOL	TYPICAL DESCRIPTION	LABORATORY CLASSIFICATION CRITERIA			
COARSE-GRAINED SOILS (MORE THAN HALF BY WEIGHT LARGER THAN 75µm)	GRAVELS MORE THAN HALF COARSE GRAINS LARGER THAN 4.75 mm	CLEAN GRAVELS (LITTLE OR NO FINES)	GW		WELL GRADED GRAVELS, GRAVEL - SAND MIXTURES, LITTLE OR NO FINES	Determine percentages of gravel and sand from grain size curve. Depending on percentages of gravel and sand, coarse grained soils are classified as follows: Less than 5% GW, GP, SW, SP More than 12% GM, GC, SM, SC Borderline cases requiring use of dual symbols	$C_u = \frac{D_{60}}{D_{10}} > 4$; $C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}} = 1 \text{ to } 3$		
			GP		POORLY GRADED GRAVELS, GRAVEL-SAND MIXTURES, LITTLE OR NO FINES		NOT MEETING ALL GRADATION REQUIREMENTS FOR GW		
		GRAVELS WITH FINES (APPRECIABLE AMOUNT OF FINES)	GM		SILTY GRAVELS, GRAVEL-SAND-SILT MIXTURES		ATTERBERG LIMITS BELOW "A" LINE I_p LESS THAN 4		
			GC		CLAYEY GRAVELS, GRAVEL-SAND-CLAY MIXTURES		ATTERBERG LIMITS ABOVE "A" LINE I_p MORE THAN 7		
	SANDS MORE THAN HALF COARSE GRAINS SMALLER THAN 4.75 mm	CLEAN SANDS (LITTLE OR NO FINES)	SW		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		POORLY GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES		NOT MEETING ALL GRADATION REQUIREMENTS FOR SW		
		SAND WITH FINES (APPRECIABLE AMOUNT OF FINES)	SM		SILTY SANDS, SAND-SILT MIXTURES		ATTERBERG LIMITS BELOW "A" LINE I_p LESS THAN 4		
			SC		CLAYEY SANDS, SAND-CLAY MIXTURES		ATTERBERG LIMITS ABOVE "A" LINE I_p 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			INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTY OR CLAYEY FINE SANDS OR CLAYEY SILTS WITH SLIGHT PLASTICITY	CLASSIFICATION IS BASED UPON PLASTICITY CHART (see below)
				$w_L > 50\%$	MH			INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS, FINE SANDY OR SILTY SOILS	
CLAYS ABOVE "A" LINE NEGLECTIBLE ORGANIC CONTENT	$w_L < 30\%$		CL		INORGANIC CLAYS OF LOW PLASTICITY, GRAVELLY, SANDY, OR SILTY CLAYS, LEAN CLAYS				
	$30\% < w_L < 50\%$		CI		INORGANIC CLAYS OF MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS				
	$w_L > 50\%$		CH		INORGANIC CLAYS OF HIGH PLASTICITY, FAT CLAYS				
ORGANIC SILTS & CLAYS BELOW "A" LINE	$w_L < 50\%$		OL		ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW AND MEDIUM PLASTICITY				
	$w_L > 50\%$		OH		ORGANIC CLAYS OF HIGH PLASTICITY, ORGANIC SILTS				
HIGHLY ORGANIC SOILS			Pt		PEAT AND OTHER HIGHLY ORGANIC SOILS	STRONG COLOR OR ODOR, AND OFTEN FIBROUS TEXTURE			

SPECIAL SYMBOLS

	BEDROCK (UNDIFFERENTIATED)		OVERBURDEN (UNDIFFERENTIATED)
	SANDSTONE		SILTSTONE
	CLAYSTONE (CLAYSHALE OR MUDSTONE)		
	LIMESTONE		
	CONGLOMERATE		
	COAL		



THURBER ENGINEERING LTD.

MODIFIED UNIFIED CLASSIFICATION SYSTEM FOR SOILS

(MODIFIED BY PFRA, 1985)

CLIENT: DUB ARCHITECTS LTD.		PROJECT: Rossdale Centre Phase 1 - Geo Inv		BOREHOLE NO: TH16-01	
DRILLING COMPANY: ALL SERVICE DRILLING INC		DATE DRILLED: April 20, 2016		PROJECT NO: 12553	
DRILL/METHOD: M10 / Solid Stem Augers		LOCATION: See Drawing #12553-1		ELEVATION:	
SAMPLE TYPE		☐ GRAB SAMPLE ☒ SPT			
BACKFILL TYPE		☒ BENTONITE ☐ DRILL CUTTINGS ☐ SLOUGH			

DEPTH (m)	SAMPLE TYPE	SPT (N)	▲ CPEN (kPa)▲ 50 100 150 200 ■ SPT Blows/300 mm■ 10 20 30 40 PLASTIC M.C. LIQUID 10 20 30 40		REMARKS	USC	SOIL SYMBOL	SOIL DESCRIPTION	DEPTH (m)
0							OL	TOPSOIL black, silty, some clay, trace rootlets and coarse gravel	0
1	9						CH	CLAY stiff, brown, silty	1
2							CI	-very silty	2
3	8						CI	-trace calcareous deposits	3
4							CL		4
5	5						CL	-firm	5
6							CL	-light brown	6
7	9						CL-ML	-stiff, sandy, trace fine gravel	7
8							SP-SM	SAND grey, fine grained, silty, some clay	8
9	37						GM	GRAVEL dense, grey, silty, trace clay	9
10							GC	-trace cobbles	10
							GM		
							GC	-trace cobbles	
								CLAY SHALE, very hard, grey, silty, some siltstone fragments	

 THURBER ENGINEERING LTD.		FIELD LOGGED BY: GM PREPARED BY: JGP REVIEWED BY: HSP	COMPLETION DEPTH: 10.4 m COMPLETION DATE: 4/20/16
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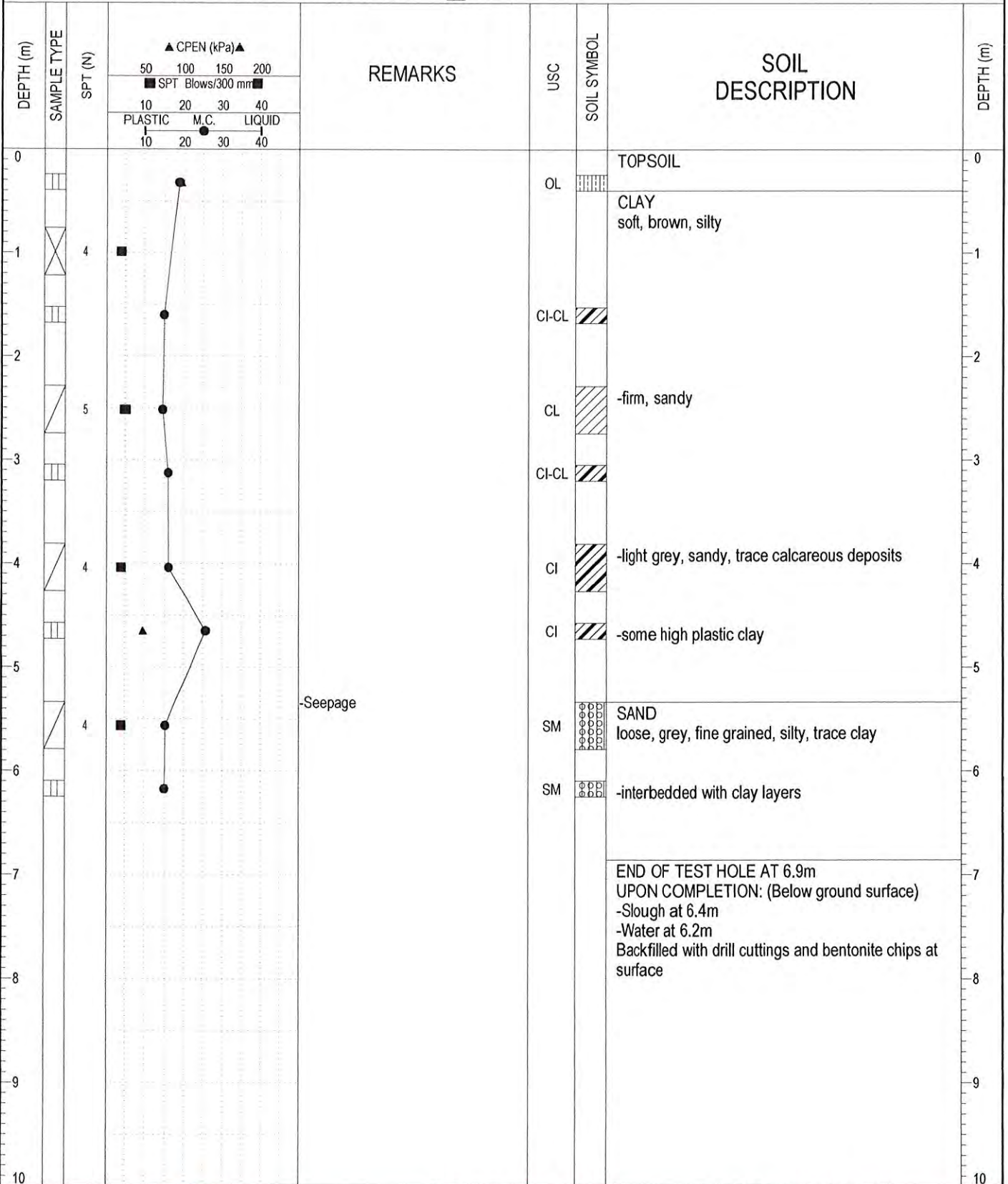
CLIENT: DUB ARCHITECTS LTD.		PROJECT: Rosssdale Centre Phase 1 - Geo Inv		BOREHOLE NO: TH16-01	
DRILLING COMPANY: ALL SERVICE DRILLING INC		DATE DRILLED: April 20, 2016		PROJECT NO: 12553	
DRILL/METHOD: M10 / Solid Stem Augers		LOCATION: See Drawing #12553-1		ELEVATION:	
SAMPLE TYPE		☐ GRAB SAMPLE ☒ SPT			
BACKFILL TYPE		☒ BENTONITE ☒ DRILL CUTTINGS ☐ SLOUGH			

DEPTH (m)	SAMPLE TYPE	SPT (N)	▲ CPEN (kPa) ▲ 50 100 150 200 ■ SPT Blows/300 mm ■ 10 20 30 40 PLASTIC M.C. LIQUID 10 20 30 40	REMARKS	SLOTTED PIEZOMETER	USC	SOIL SYMBOL	SOIL DESCRIPTION	DEPTH (m)
10		52		>> Cpen > 215kPa		CS		CLAY SHALE - CONTINUED	10
11								END OF TEST HOLE AT 10.4m UPON COMPLETION: (Below ground surface) -Slough at 7.5m Standpipe piezometer installed WATER LEVEL BELOW GROUND SURFACE: -April 25, 2016 = 6.8m	11
12									12
13									13
14									14
15									15
16									16
17									17
18									18
19									19
20									20

 THURBER ENGINEERING LTD.	FIELD LOGGED BY: GM	COMPLETION DEPTH: 10.4 m
	PREPARED BY: JGP	COMPLETION DATE: 4/20/16
	REVIEWED BY: HSP	Page 2 of 2

CLIENT: DUB ARCHITECTS LTD.	PROJECT: Rossdale Centre Phase 1 - Geo Inv	BOREHOLE NO: TH16-02
DRILLING COMPANY: ALL SERVICE DRILLING INC	DATE DRILLED: April 20, 2016	PROJECT NO: 12553
DRILL/METHOD: M10 / Solid Stem Augers	LOCATION: See Drawing #12553-1	ELEVATION:

SAMPLE TYPE	☐ GRAB SAMPLE	☒ NO RECOVERY	☒ SPT
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BOREHOLE LOG 12553.GPJ THRB AB.GDT 5/11/16- LIBRARY-NEW LOGO.GLB



THURBER ENGINEERING LTD.

FIELD LOGGED BY: GM

PREPARED BY: JGP

REVIEWED BY: HSP

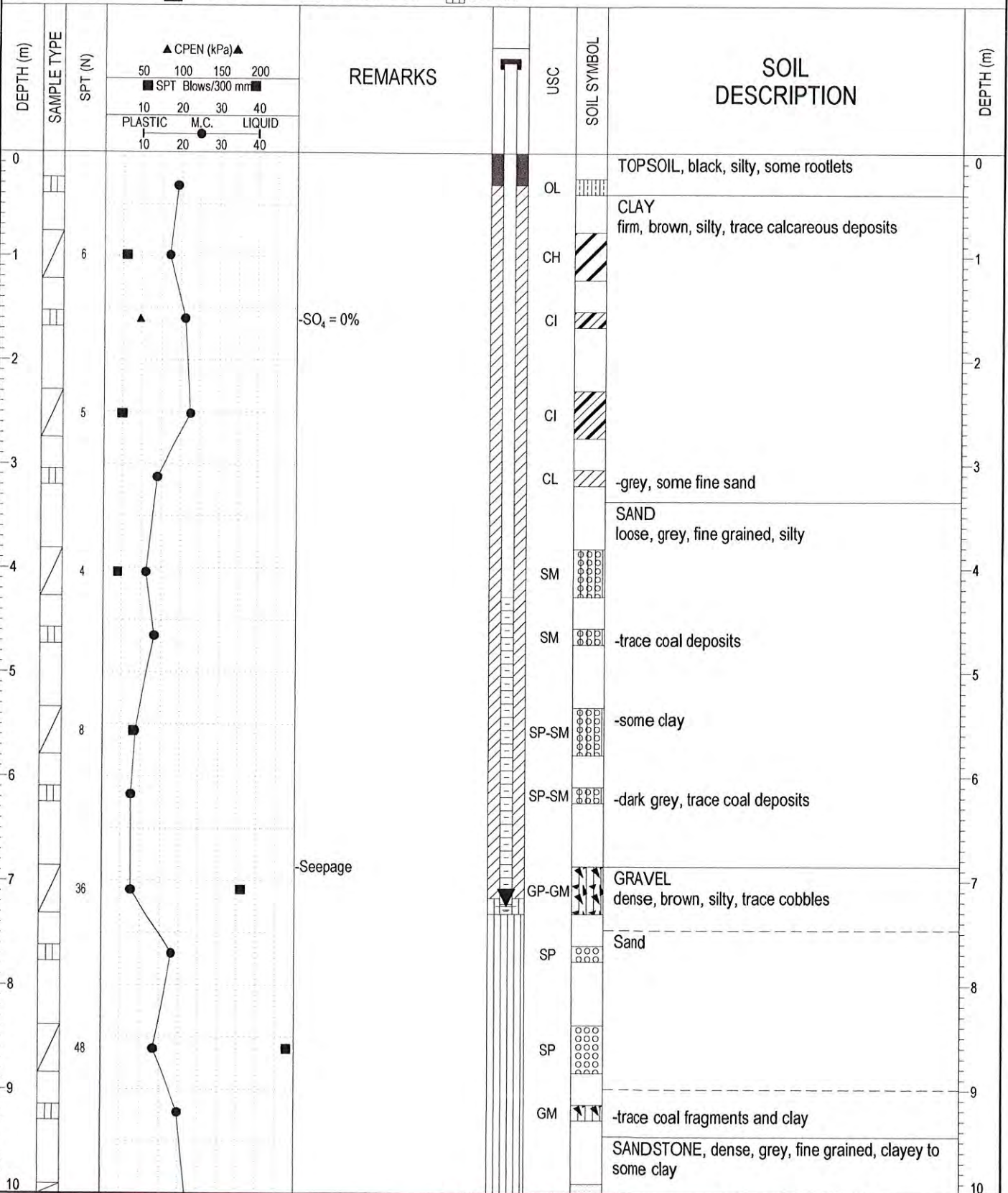
COMPLETION DEPTH: 6.9 m

COMPLETION DATE: 4/20/16

Page 1 of 1

CLIENT: DUB ARCHITECTS LTD.	PROJECT: Rosedale Centre Phase 1 - Geo Inv	BOREHOLE NO: TH16-03
DRILLING COMPANY: ALL SERVICE DRILLING INC	DATE DRILLED: April 20, 2016	PROJECT NO: 12553
DRILL/METHOD: M10 / Solid Stem Augers	LOCATION: See Drawing #12553-1	ELEVATION:

SAMPLE TYPE	☐ GRAB SAMPLE	☒ SPT	
BACKFILL TYPE	☒ BENTONITE	☒ DRILL CUTTINGS	☐ SLOUGH



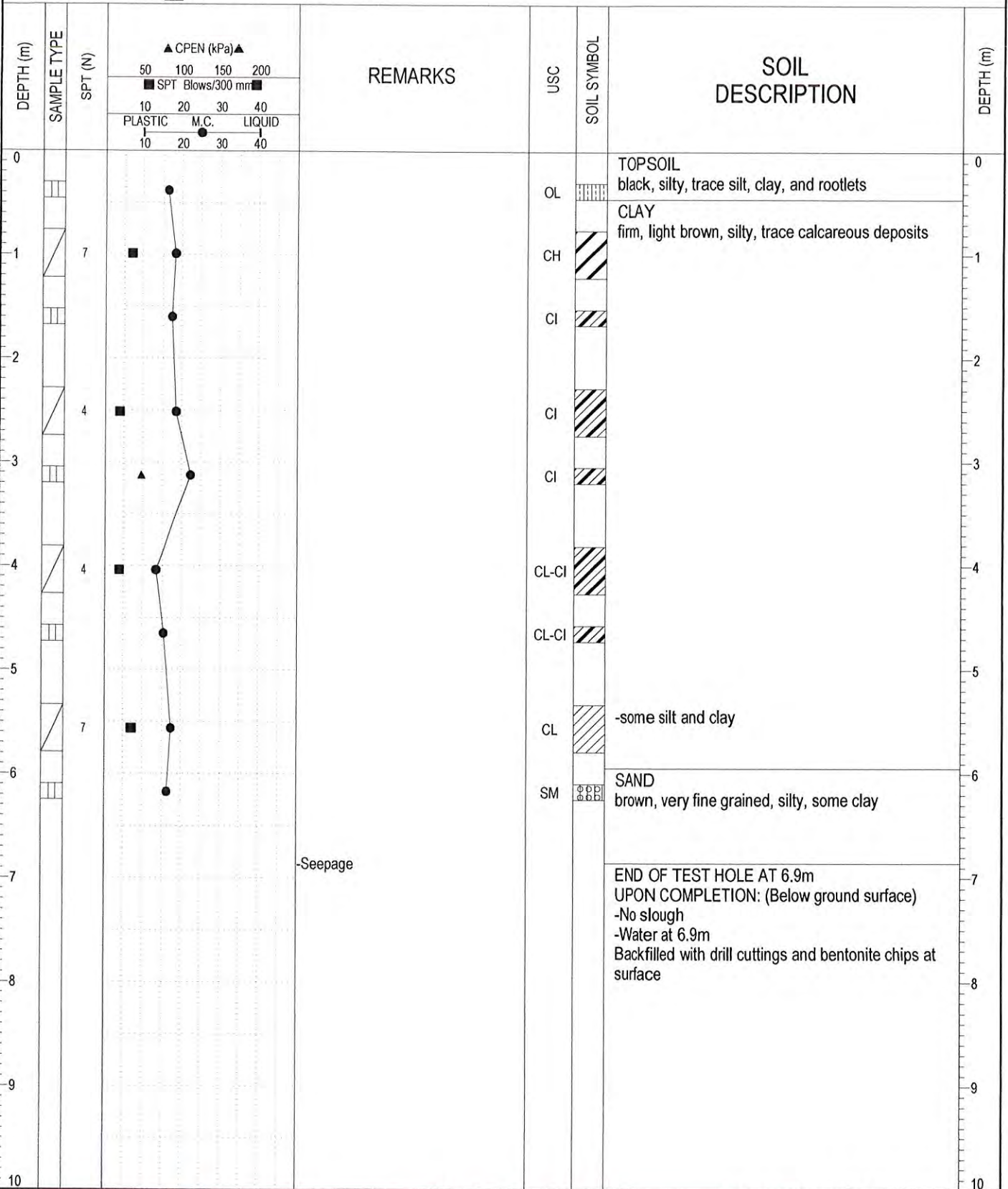
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DRILLING COMPANY: ALL SERVICE DRILLING INC		DATE DRILLED: April 20, 2016		PROJECT NO: 12553	
DRILL/METHOD: M10 / Solid Stem Augers		LOCATION: See Drawing #12553-1		ELEVATION:	
SAMPLE TYPE		☐ GRAB SAMPLE ☒ SPT			
BACKFILL TYPE		☒ BENTONITE ☒ DRILL CUTTINGS ☐ SLOUGH			

DEPTH (m)	SAMPLE TYPE	SPT (N)	▲ CPEN (kPa) ▲		REMARKS	SLOTTED PIEZOMETER	USC	SOIL SYMBOL	SOIL DESCRIPTION	DEPTH (m)		
			50	100							150	200
			■ SPT Blows/300 mm ■									
			10	20							30	40
			PLASTIC M.C. LIQUID									
			10	20	30	40						
10		35						SS	SANDSTONE - CONTINUED	10		
11									END OF TEST HOLE AT 10.4m UPON COMPLETION: (Below ground surface) -Slough at 7.2m -Water at 7.0m Standpipe piezometer installed WATER LEVEL BELOW GROUND SURFACE: -April 25, 2016 = 7.2m	11		
12										12		
13										13		
14										14		
15										15		
16										16		
17										17		
18										18		
19										19		
20										20		

 THURBER ENGINEERING LTD.	FIELD LOGGED BY: GM	COMPLETION DEPTH: 10.4 m
	PREPARED BY: JGP	COMPLETION DATE: 4/20/16
	REVIEWED BY: HSP	Page 2 of 2

CLIENT: DUB ARCHITECTS LTD.	PROJECT: Rosedale Centre Phase 1 - Geo Inv	BOREHOLE NO: TH16-04
DRILLING COMPANY: ALL SERVICE DRILLING INC	DATE DRILLED: April 20, 2016	PROJECT NO: 12553
DRILL/METHOD: M10 / Solid Stem Augers	LOCATION: See Drawing #12553-1	ELEVATION:

SAMPLE TYPE ☐ GRAB SAMPLE ☒ SPT



BOREHOLE LOG 12553.GPJ THRB, AB, GDT 5/11/16- LIBRARY-NEW LOGO.GLB



THURBER ENGINEERING LTD.

FIELD LOGGED BY: GM

PREPARED BY: JGP

REVIEWED BY: HSP

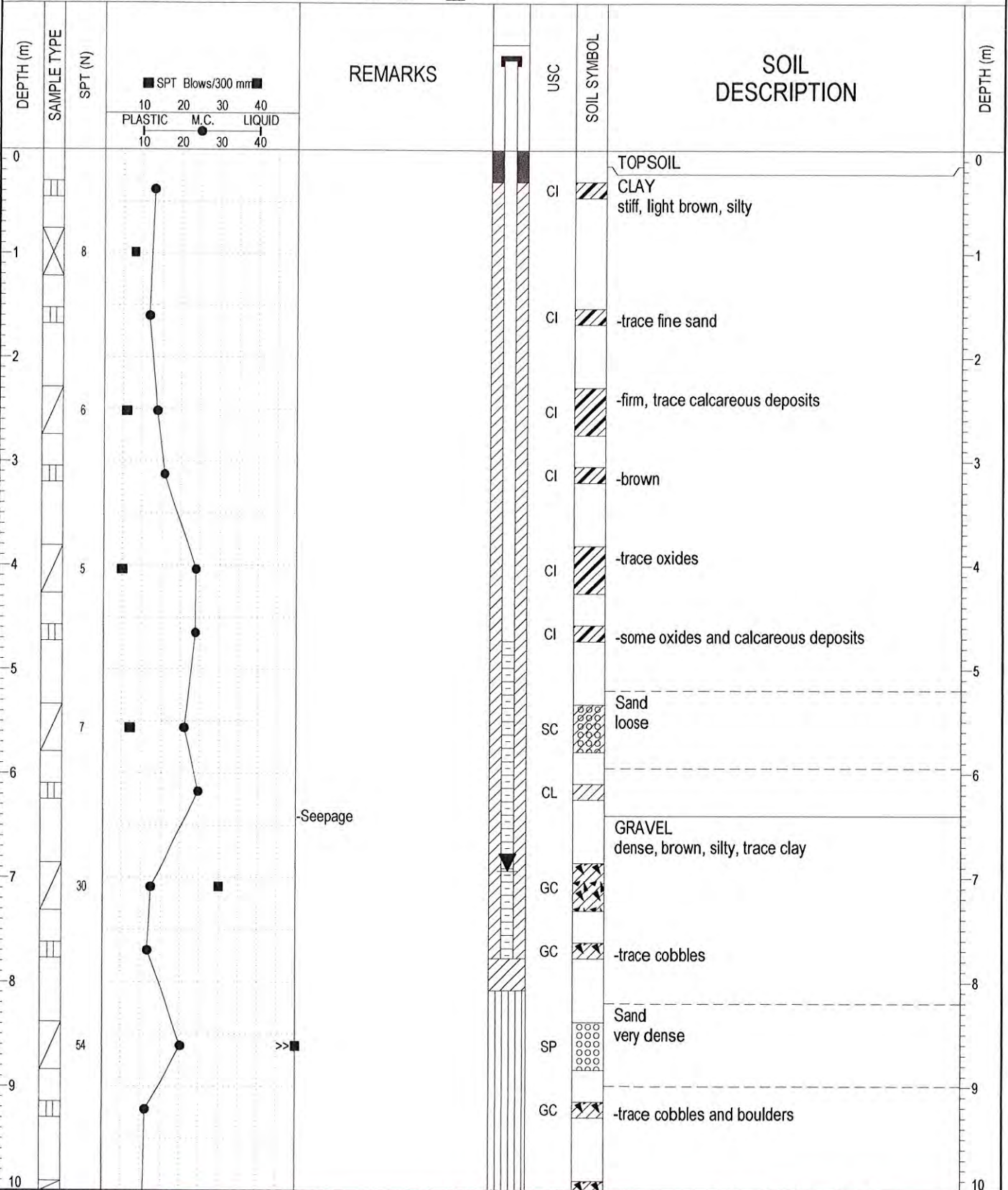
COMPLETION DEPTH: 6.9 m

COMPLETION DATE: 4/20/16

Page 1 of 1

CLIENT: DUB ARCHITECTS LTD.	PROJECT: Rossdale Centre Phase 1 - Geo Inv	BOREHOLE NO: TH16-05
DRILLING COMPANY: ALL SERVICE DRILLING INC	DATE DRILLED: April 20, 2016	PROJECT NO: 12553
DRILL/METHOD: M10 / Solid Stem Augers	LOCATION: See Drawing #12553-1	ELEVATION:

SAMPLE TYPE	☐ GRAB SAMPLE	☒ NO RECOVERY	☒ SPT
BACKFILL TYPE	☒ BENTONITE	☒ DRILL CUTTINGS	☐ SLOUGH



BOREHOLE LOG 12553.GPJ THRB AB.GDT 5/11/16- LIBRARY-NEW LOGO.GLB



THURBER ENGINEERING LTD.

FIELD LOGGED BY: GM

PREPARED BY: JGP

REVIEWED BY: HSP

COMPLETION DEPTH: 10.4 m

COMPLETION DATE: 4/20/16

Page 1 of 2

CLIENT: DUB ARCHITECTS LTD.		PROJECT: Rosedale Centre Phase 1 - Geo Inv		BOREHOLE NO: TH16-05	
DRILLING COMPANY: ALL SERVICE DRILLING INC		DATE DRILLED: April 20, 2016		PROJECT NO: 12553	
DRILL/METHOD: M10 / Solid Stem Augers		LOCATION: See Drawing #12553-1		ELEVATION:	
SAMPLE TYPE		☐ GRAB SAMPLE ☒ NO RECOVERY ☒ SPT			
BACKFILL TYPE		☒ BENTONITE ☒ DRILL CUTTINGS ☐ SLOUGH			

DEPTH (m)	SAMPLE TYPE	SPT (N)	REMARKS	SLOTTED PIEZOMETER	USC	SOIL SYMBOL	SOIL DESCRIPTION	DEPTH (m)
10		28			GC		GRAVEL - CONTINUED -compact	10
11							END OF TEST HOLE AT 10.4m UPON COMPLETION: (Below ground surface) -Slough at 8.1m -Water at 7.0m Standpipe piezometer installed WATER LEVEL BELOW GROUND SURFACE: -April 25, 2016 = 6.9m	11
12								12
13								13
14								14
15								15
16								16
17								17
18								18
19								19
20								20

BOREHOLE LOG 12553.GPJ TH16R AB.GDT 5/11/16- LIBRARY-NEW LOGO.GLB



THURBER ENGINEERING LTD.

FIELD LOGGED BY: GM
PREPARED BY: JGP
REVIEWED BY: HSP

COMPLETION DEPTH: 10.4 m
COMPLETION DATE: 4/20/16