McINTOSH•LALANI
ENGINEERING LTD.

M•L 8855

Geotechnical Report

Proposed Drumheller Curling Club

Drumheller, Alberta

Prepared For

GEC Architecture

Submitted On

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1.0 INTRODUCTION

This report presents the results of a geotechnical evaluation conducted by McIntosh•Lalani Engineering Ltd. (M•L) for a proposed construction of a new curling club building to be located adjacent to the existing Drumheller Memorial Arena in Drumheller, Alberta. A total of eight Boreholes (BH's 1-8) were advanced within the proposed development area. The objective of this evaluation was to assess the general subsurface soil and groundwater conditions at the site for the design and construction of the proposed curling club building. This evaluation was undertaken at the request of Mr. Robert Sterling of GEC Architecture.

This report presents the results of the drilling program and provides geotechnical recommendations for construction.

1.1 PROJECT AND SITE DETAILS

The proposed development at the site will include the design and construction of a 4 to 6 sheet curling club building with a partial second floor to oversee the ice surface. The proposed building will be a clear-span with wood or steel construction. A refrigerated slab has been proposed to be designed for year round operation. No below-grade development has been proposed for the subject site. The building site has been selected, however two options are being considered for the building orientation; a north-south orientation, and an east-west orientation. The site is located north of the existing Drumheller Memorial Arena and north of the intersection of 1st Avenue West and 1st Street West in Drumheller, AB. The site is bounded by an existing skate park / outdoor pool to the west, Aquaplex swimming facility further to the west and south bank of the Red Deer River to the north. There is currently no development present on the east side of the subject site.

2.0 METHODOLOGY

In order to assess the geotechnical site conditions including soil stratigraphy, groundwater conditions and soil properties, M•L completed a program of borehole drilling and installation of standpipe well combined with laboratory index testing.

The borehole location was selected by representatives of M•L. The location of the boreholes are illustrated in Drawing 8855.00.G101. The borehole logs are presented in Appendix A.

2.1 SOILS INVESTIGATION

The subsurface investigation consisted of advancing a total of eight (8) boreholes within the area for the proposed development to depths ranging from of 9.1 to 15.2 metres below the existing grade. The boreholes were advanced on September 14, 2018 using a track mounted solid stem auger drill rig contracted from All Service Drilling of Airdrie, Alberta. Classification of the soil was done from the disturbed samples obtained from the auger flights and from the Standard Penetration Test (SPT) operation. SPT blow counts were utilized to aid in determining in-situ soil strengths.

2.2 GROUNDWATER

Upon completion of the boreholes, the depth of the borehole was measured, including any slough, and the presence or absence of free water within the borehole was noted. A 25 mm diameter slotted PVC standpipe well was installed in the boreholes to allow future measurement of groundwater level within the depth of the investigation. This type of installation assumes a simple groundwater regimen. Specifics of the well installation are illustrated on the borehole log.

2.3 LABORATORY TESTING

Laboratory testing including natural moisture content, soluble sulphate and hydrometer grain size analysis testing has been completed and the results have been reported in a separate letter.

3.0 SUBSURFACE CONDITIONS

At the time this report was prepared, information on subsurface stratigraphy was available only at discrete borehole locations. Conditions were extrapolated and interpolated from the borehole locations to develop recommendations. Adequate monitoring should be provided during construction to check that these assumptions are reasonable. The below summarizes the subgrade conditions encountered in the drilling program. More detailed soil description is contained in the borehole logs in Appendix A.

3.1 SURFICIAL GEOLOGY

The site is located near the south bank of the Red Deer River. The expected soil profile in the area consists of mostly quaternary fluvial (stream deposits) including gravel, sand, silt and clay. These deposits are overlying Cretaceous or Tertiary shale with interbedded sandstone and coal bedrock. These two units could be separated by glacial till deposits associated with the Classical Wisconsin Laurentide glaciation (A. Stalker, GSC Memoir 370, 1972).

3.2 SOILS

Asphalt approximately 50 mm in thickness was logged at the surface of Boreholes # 3 to 6. Topsoil approximately 400 to 450 millimetres in thickness was encountered at the surface of Boreholes # 1, 2, 7 & 8. Loams and silty sand fills were encountered at the site to depths ranging from 0.3 to 2.0 metres below grade in majority of the site area, however, at the location of Borehole # 2, fill with coal inclusion and rubble was encountered to a depth of 3.7 metres below grade. Silty sand and silty clay were encountered below fills at the site. A highly weathered and weak bedrock was encountered in Borehole # 5 at a depth of 11.3 metres below grade. No bedrock was logged up to depth of 15.2 metres below grade in Borehole # 2.

Note: Because surficial organic soils are affected by the erosive forces of wind and precipitation, and are redistributed by agricultural practices, the thicknesses of these soil layers can vary widely. As such, the thickness of topsoil organic browns as measured in the boreholes should not be relied upon to make estimates of stripping quantities.

Based on the soil logged in the borehole opened, the soil at the site in general consisted of:

3.2.1 Fills / loams

The site has encountered significant fill soils. Below topsoil in Boreholes # 7 & 8, a layer of black loamy soil was logged at 0.45 metres and extended to depths of 0.9 to 1.5 metres below grade. In majority of the boreholes, the silty sand fill extended to depths ranging from 0.3 to 1.2 metres below grade, however at the location of Borehole # 2, the depth of fill was 3.7 metres and at Borehole # 8, the fill below loam was 2.0 metres below grade. Trace amounts of gravel and rubble were noted within the fill soils. The deposit was observed to be of loose relative density. The fill was damp and medium brown in colour.

3.2.2 Silty Sand

Silty sand was encountered below fills and below sandy gravel (see below) at the borehole locations and extended to depths of 3.3 to 9.1 metres below existing grade. The deposit was observed to be of very loose to loose relative density as adjudged by standard penetration blow counts of 4 to 10 blows per 30 centimetres. The silty sand was damp to moist and light brown to medium brown in colour.

3.2.3 Sandy Gravel

Sandy gravel was logged in all boreholes at depths ranging from 3.3 to 4.3 metres below grade and extended to depths of 7.0 to 11.3 metres below grade. The deposit was observed to be of loose to compact relative density. The sandy gravels were damp to wet and generally medium brown in colour.

3.2.4 Silty Clay

Silty clay was logged below gravels in some of the boreholes and extended to depths ranging from 9.1 to 15.2 metres below grade below grade. The deposit was generally observed to be of firm to very stiff consistency. The clays were generally low plastic, moist to wet and medium brown to dark grey in colour.

3.2.5 Bedrock

Highly weathered shale and coal were encountered in Borehole # 5 at a depth of 11.3 metres below grade, however, no bedrock was encountered up to a depth of 15.2 metres below grade in Borehole # 2 advanced at the site. Standard penetration blow counts of 41 and 47 blows per 30 centimetres were recorded within the bedrock. The bedrock was light brown to dark grey in colour and generally damp to wet.

For more detailed soil profiles refer to the borehole logs attached.

3.3 GROUNDWATER

Groundwater seepage was encountered during monitoring and the boreholes were noted to be wet upon completion. In the boreholes, groundwater level was measured and recorded in the standpipe piezometers on September 25, 2018, at which time groundwater was detected at depths of 4.57 to 11.58 metres below ground surface. Groundwater levels fluctuate seasonally and in early September are expected to be at or near their seasonal peak in an average year.

4.0 DISCUSSION AND RECOMMENDATIONS

4.1 GENERAL

The subsurface soil conditions encountered at the site are extremely variable. Variable fills to varying depths were logged at the site. Bedrock in Borehole # 5 was logged at a depth of 11.3 metres and no bedrock was logged to a depth of 15.2 metres below grade in Borehole # 2. Topsoil, loams, fills and other deleterious material encountered at the site are not suitable to support the foundations and curling club rink slab. Deep pile foundation system and structural slab are recommended for the proposed development. The subsurface conditions below fills at the site are considered suitable for the proposed development. The geotechnical considerations at the site are summarized below:

- A deep pile foundation system such as drilled cast-in-place concrete piles or steel H-Piles driven to practical refusal within the bedrock logged at a depth of 11.5 metres below grade (in Borehole # 5, and to be confirmed in remaining areas) are feasible for the soil conditions. Bedrock depth must be confirmed during the installation or discuss variability in bedrock depth in area, known to be 11 to 15 metres in a short distance. Groundwater will likely be encountered during drilling and casing will likely be required due to the cohesionless nature of silty / sandy and gravelly soils present at the site.
- The existing fill soils are not suitable for a slab-on-grade design and structural slab supported on piles is recommended due to the differential movements that can occur in a proposed refrigerated curling slab. Slab-on-grade construction may be possible after removing the existing topsoil/loam and fill soils to an approximate depth of 2 metres and replacing the soils using well graded sandy gravels in control compacted and tested layers. After removing 2 metres of existing fill, the upper 0.3 to 0.6 metres (depending upon the disturbance to predominantly silty sand soils) shall be densified and compacted prior to placing the new fills. The new fills should be placed in layers not exceeding 200 millimetres compacted thickness and shall be compacted to 100 percent of Standard Proctor Maximum Dry Density (SPMDD). Compaction should be carried out in such a way that the result is a uniformly compacted fill layer. Existing soils after removing 2 metres of fill at the base of the excavation shall be inspected and where poor quality soils are present, undercutting and replacement will be necessary.

- Temporary excavations will need to be constructed with minimum backsloping at a gradient of 1 Horizontal to 1 Vertical (1H:1V). Additional backsloping may be required if excessive sloughing or groundwater infiltration is experienced.
- Groundwater was encountered at depths of approximately 4.5 to 5.5 metres below existing grade in majority of the boreholes. Below grade development is not anticipated for this project, minor seepage into construction excavations may need to be dewatered using a system of ditches, sumps and pumps. For drilled piles, seepage may likely be encountered within the sand and silt seams present at the site and the contractor should have casing, tremie pipes and cleaning buckets available. A subsurface weeping tile is required for any below grade development.

This list should not be considered all-inclusive and should be read in conjunction with the remainder of this report. Geotechnical foundation design parameters, slab-on-grade recommendations, groundwater concerns, and additional construction recommendations are provided in the sections below.

4.2 FOUNDATION DESIGN RESISTANCE FACTORS

Load and Resistance Factor Design (LRFD) parameters are presented below for shallow and deep foundation design. Ultimate Limit State (ULS) resistances are presented, and should be utilized with the following design formula, as per the Canadian Foundation Manual Fourth Edition 2006:

$$\Phi R_n \geq \Sigma \alpha_i S_{ni}$$

Where:

Φ = Geotechnical resistance factor

R_n = Nominal (ultimate) geotechnical resistance

α_i = Load safety factor determined by structural engineer

S_{ni} = Specified load component

i = Represents various types of loads

The values for load factors (α_i), geotechnical resistance factor (Φ) and load combinations are specified by applicable codes e.g. NBCC. As per the NBCC, we recommend use of the following Φ:

Table 1: Geotechnical Resistance Factors for Foundations

Description		Resistance Factor (Φ)
Shallow Foundations – vertical bearing resistance, semi-empirical analysis		0.5
Shallow Foundation – sliding		0.8
Deep Foundations		
Resistance to compressive axial load	from semi-empirical analysis	0.4
	from static loading test results	0.6
	from dynamic monitoring results (i-e, pile driving analyzer (PDA) testing	0.5
Uplift resistance	from semi-empirical analysis	0.3
	from loading test results	0.4
Horizontal Load resistance		0.5

4.3 DRILLED CAST-IN-PLACE CONCRETE PILES

Drilled cast-in-place concrete piles may be designed to resist axial compressive loads on the basis of allowable skin friction and end-bearing parameters given below. End-bearing should, however, not be taken into account for small diameter (less than 600 mm shaft diameter) piles because of the difficulties associated with ensuring a clean base.

For large diameter piles (shaft diameter 600 mm or greater), end bearing may be considered in the design if facilities are available during construction for downhole cleaning and inspection.

Drilled piles should have an overall length below finished grade of not less than 5 metres and a shaft diameter of not less than 400 mm. Pile spacing should not be closer than 2.5 pile diameters measured centre to centre. An increase capacity in the piles of 33% may be used for dynamic loading.

Under-reamed (Belled) bored cast-in-place concrete piles are not possible for this site, due to the presence of sands and gravels at the site. Casing the silty / sandy and gravelly soils will likely be necessary as these layers will be prone to sloughing. Seepage is expected within these layers. The feasibility of drilled piles should be verified, due to the likely use of casing.

Pile installation monitoring during the installation process of the piles should be undertaken to verify that the encountered soils are in accordance with the soils assumed in the design. The inspections need to be completed by the geotechnical engineer of record or their representative.

The contractor should be prepared to case all pile holes as required. Some precautions should be taken during installation:

- Contractor should have casing onsite to stabilize the hole as sloughing may become a problem.
- Contractor should have tremie pipes onsite or access to a concrete boom pump truck to allow placement of concrete through water if seepage occurs.
- Contractor should have appropriate sizes of cleaning buckets onsite to prepare the base properly.

The piles may be designed for a combination of end bearing and skin friction with the understanding that some movement of the pile is required before both skin friction and end bearing support are fully mobilized.

Pile design resistance parameters are shown in Table 4 and Table 5.

4.3.1 Pile Design Using LRFD

Table 2: ULS Pile Resistances for LRFD

Soil	Approx. Depth Below Existing Grade (m)	Unfactored Ultimate Skin Friction (kPa), R_n	Unfactored Ultimate End Bearing (kPa), R_n
Surficial Soils, Fill Soils	0 – 1.5 or to depth of fills	0	---
Silty Sand / Sandy Gravels	1.5 – 11.3	45	---
Bedrock	11.3 – 15.5	195	1950

Note: Geotechnical resistance factors for uplift and compression presented in Section 4.2 of this report should be applied to the ULS design values presented above.

The Serviceability Limit States (SLS) condition should be checked upon design of the foundation loads and pile sizes and depths.

4.3.2 Pile Design Using WSM

To undertake the foundation design using the Working Stress Method, the following allowable skin friction and end bearing parameters are provided:

Table 3: Allowable Pile Bearing Resistances for WSM

Soil	Approx. Depth Below Existing Grade (m)	Allowable Skin Friction (kPa)	Allowable End Bearing (kPa)
Surficial Soils, Fill Soils	0 – 1.5 or to depth of fills	---	---
Silty Sand / Sandy Gravels	1.5 – 11.3	15	---
Bedrock*	11.3 – 15.5*	60	600

* Top of bedrock will vary in depth across the site and must be verified prior or during the course of pile installation.

If the piles are designed for depths greater than 15.5 metres, the continuity of strata shall be verified to a depth of no less than three pile diameters below the general basing elevations of the piles. The continuity of the strata may be verified by the piling contractor by over-drilling one or more piles during the course of pile installation, as appropriate, depending upon the design installation depths used.

4.3.3 Lateral Load Soil Parameters

Detailed design of laterally loaded piles should be done using a non-linear Lateral Pile Response Model that also models eccentricity of axial loading due to lateral deflection (P-Δ effects). M•L can provide these services if requested. For preliminary lateral pile design, the coefficient of horizontal subgrade reaction which is a function of pile diameter has been calculated using the Davisson, 1970 method referenced in the Canadian Foundation Manual, 3rd Edition. The recommended values are presented in the table below where D is the pile diameter.

Table 4: Horizontal Moduli of Subgrade Reaction

Soil Type	Horizontal Modulus of Subgrade Reaction (Mpa/m)	
	Sustained Loading	Cyclic Loading
Fill Soils	4/D	2/D
Silty Sand and Sandy Gravel	12/D	6/D
Bedrock	40/D	20/D

4.3.4 Group Effects

Upon completion of the pile design, M•L should review the pile layout to ensure there are no pile group effects which will impact the design capacity of the piles.

4.4 SLIDING PARAMETERS

The unfactored ultimate limit state (ULS) coefficient of friction may be taken as 0.40. It is recommended that a geotechnical resistance factor of 0.8 be applied to the unfactored ULS Coefficient of friction as specified by the NBCC 2005.

4.5 SITE SOIL CLASSIFICATION FOR SEISMIC SITE RESPONSE

Based on the soil conditions logged at the site during this investigation, the site classification for seismic site response is considered to be "Site Class D" as per Section 4.1.8.4 of the 2014 Alberta Building Code.

4.6 SETTLEMENTS

4.6.1 Pile Foundations

The settlements in the piles will depend on the pile loads, sizes and group effects. Providing the pile tips are installed to near same elevation (within a few meters) and all piles are installed as per installation requirements and the piles are designed as per the recommendations in this report, the piles should undergo relatively uniform movement. The movement required to mobilize the skin friction along the pile shaft will be approximately 1% of the pile diameter. Additional elastic shortening of the pile should be expected. Using the design values in this report the total consolidation at the pile tip should be limited to 25mm.

In addition, both differential settlements and total settlements would also depend on the settlement needed to mobilize the required geotechnical resistance which is dependent on the effective cleaning of the base of the drilled shaft.

4.7 FLOOR SLABS

The existing fill soils on this site are not suitable for construction of slab-on-grade. A structural slab supported on piles may be considered for the site. Gravel for underslab gas depressurization systems must meet the gradation specified in the Alberta Building Code section 9.13.4. The Class IA material (drainage gravel) is a suitable material for this application, however any gravel that meets the specified gradation as well as any structural requirements may be used. An acceptable gravel will have less than 10% of the material passing a 4 mm sieve. Granular base (25 mm crush) gravel generally does not meet this requirement.

4.7.1 Slab-on-Grade

Fills were logged generally to depths of 1.2 to 2.0 metres and to 3.7 metres below grade locally in Borehole # 2 at the site. Slab-on-grade construction is considered feasible provided certain precautions are undertaken and after removing the existing topsoil/loam and fill soils to an approximate depth of 2 metres and replacing the soils by using well graded sandy gravels in control compacted and tested layers. After removing 2 metres of existing fill, the upper 0.3 to 0.6 metres (depending upon the disturbance to predominantly silty sand soils) shall be densified and compacted prior to placing the new fills. The new fills should be placed in layers not exceeding 200 millimetres compacted thickness and shall be compacted to 100 percent of Standard Proctor Maximum Dry Density (SPMDD). Existing soils after removing 2 metres of fill at the base of the excavation shall be inspected and where poor quality soils are present, undercutting and replacement will be necessary.

Some relative movement between floor slab-on-grade and adjacent walls or foundations and differential movements within slab should be anticipated. Generally, if the recommendations outlined in this report are followed, these movements should be acceptably small. It is possible, however, that some cracking of the slab or distortion of any internal partition walls supported by the slab may occur. Such damage may be visible, particularly if a brittle surface finishing, such as ceramic tiles, is adopted. The risk of such damage should be weighed against the additional costs associated with alternative slab support systems, such as structurally supported slabs.

4.8 STRUCTURAL SLABS

A structurally supported floor slab system may be considered for the subject site due to deep variable fills encountered at the site and to avoid differential movement between the slab and adjacent walls or foundations.

With any structurally supported floor slab system, there is a risk of movement of the ground beneath the slab relative to the slab. This can lead to problems if piping and other utilities that are connected to the slab are embedded within the ground beneath the slab. All utilities beneath structurally supported ground floor slabs should be protected from the effects of such differential movement. This can be accomplished by placing utilities within boxes suspended from the structural slab.

4.9 FROST PROTECTION

The on-site silty soils encountered throughout the site should be considered very frost susceptible which will result in frost heave displacement in the soil when frozen.

4.9.1 Structures

For protection against frost action, perimeter footings or grade beams in heated structures should be extended to such depths as to provide a minimum soil cover of 1.4 metres. Exterior footings or grade beams in unheated structures should have a minimum soil cover of 2.1 metres, unless provided with equivalent insulation. Grade beams that do not have adequate soil cover for frost protection should have a minimum 100 mm void space on the underside of the grade beam to reduce the risk of interaction with the underlying soil. Any portion of the foundation that extends more than 1.0 metres from the heated structure should be considered to be an unheated foundation.

4.9.2 Surface Concrete

The surficial site soils are predominantly composed of frost susceptible soils. Therefore, some precautions should be followed for the design and construction of concrete flatworks at the site.

In all unheated areas, the site soils will likely experience some degree of heave due to frost formation during the winter months. Generally speaking, if proper consideration is given to the recommendations contained in Section 4.10, proper drainage will prevent the subgrade from becoming saturated and will help reduce the severity of frost heave. Nevertheless, concrete flatwork should be designed with anticipation of some frost heave occurring. Concrete sidewalks should be dowelled into footings or grade beams in threshold areas where heave of the concrete panels would obstruct the proper opening of the door and present a tripping hazard. As the outside edge of these panels will still heave, the panel should either be properly jointed to control crack locations, or reinforced by the placement of reinforcing steel 10 mm bars at a 300 mm spacing. The depth of the reinforcement should be controlled so that the reinforcement is properly located within the concrete panels.

Alternatively, rigid insulation can be placed below flatwork to prevent frost formation in the underlying subgrade. M•L can provide recommendations for such insulation if required.

4.10 SITE GRADING AND DRAINAGE

It is recommended that final site grading be provided to direct water to areas remote from the proposed structures. Minimum landscape gradients of 1.5 percent are recommended to reduce the risk of run-off ponding in localized areas. Driveways, parking areas or landscaping within a zone of approximately 2 m of the exterior perimeter of any structure should be graded to drain away from the structures at a minimum gradient of 2 percent. Furthermore, downspouts should be positively directed away from buildings.

All fill soils placed on site should consist of general engineered fill as per Section 4.19 of this report.

4.11 CONSTRUCTION EXCAVATIONS

The composition and consistencies of the soils encountered at the site are such that conventional hydraulic excavators should generally be able to remove the native soils.

All excavations should be carried out in accordance with Alberta Occupational Health and Safety (OH&S) Regulations. Excavations in the cohesionless sands and silts should have backsloping at a minimum gradient of 1H:1V from the base of excavation. If excessive sloughing is encountered, additional side sloping may be necessary. Should space constraints not allow adequate side sloping for the excavation to ensure a safe temporary excavation, a designed shoring system will be necessary. A qualified geotechnical engineer should review the excavation stability to ensure excavation safety prior to workers entering the excavation.

4.12 GROUNDWATER CONSIDERATIONS

Groundwater level was measured in the standpipe piezometer and recorded on September 25, 2018, at which time groundwater was detected at depths of 4.57 to 11.58 metres below ground surface. Groundwater levels fluctuate seasonally and in early September are expected to be at or near their seasonal peak in an average year. Groundwater is not expected to have a significant impact on most construction activities. Groundwater seepage will likely be encountered during pile installation and appropriate equipment including casing, tremie pipes and cleaning buckets should be available.

4.13 PERMANENT DEWATERING SYSTEM

Based on the soil and groundwater conditions observed at the site, weeping tile is not considered necessary for the proposed building with a structural slab constructed above the adjacent grade, however, weeping tile would generally be recommended for building with a basement.

The ground surface adjacent to the building must be graded so as to direct surface water away from the building.

4.14 CONCRETE

Testing for soluble sulphates indicates a negligible soluble sulphate concentration of up to 0.082 percent. Therefore, the use of Type GU (Normal Portland) cement concrete in accordance with CSA A23.1, Table 2 for F-2 exposure is suitable for all concrete in contact with the soil which these samples represent. The F-2 exposure class requires minimum 25 MPa strength at 28 days, a maximum water to cementing materials ratio of 0.55 and 4-7 percent entrained air by volume based on 14-20 mm aggregate. It is recommended that all imported soils to be utilized on site be tested for soluble sulphate concentrations.

4.15 LATERAL WALL PRESSURES

Permanent and temporary walls should be designed to resist all lateral pressures including those due to soil or backfill, surcharges, water and adjacent footings using the following expressions defined in terms of total and effective stresses:

$$P_{\text{lateral pressure}} = P'_{\text{earth+surcharge}} + P_{\text{net water}} + P'_{\text{adj ft}}$$

where $P_{\text{lateral pressure}}$ = total lateral pressure at a given depth (kN/m^2)

$P'_{\text{earth+surcharge}}$ = lateral earth pressure due to soil or fill and surcharges at a given depth (kN/m^2)

= $K(\gamma h + q)$ above water table or phreatic surface

= $K(\gamma' h + q)$ below water table or phreatic surface

$P_{net\ water}$ = net water pressure on wall at a given depth (kN/m^2), calculated by hand drawn flow net or computer solution based on drainage conditions

$P'_{adj\ ft}$ = lateral earth pressure due to adjacent footings at given depth (kN/m^2)

K = coefficient of lateral earth pressure, K_a, K_o, K_p or combination of as noted below

K_a = coefficient of active earth pressure

K_o = coefficient of at-rest earth pressure

K_p = coefficient of passive earth pressure

γ' = submerged unit weight of backfill or natural soil (kN/m^3)

γ' = $\gamma - \gamma_w$

γ = bulk unit weight of backfill or natural soil (kN/m^3)

γ_w = unit weight of water 9.81 kN/m^3

h = excavation depth (m)

q = surcharge load (kN/m^2)

Table 5 below presents coefficients of lateral earth pressure and unit weights.

Table 5: Coefficients of Lateral Earth Pressure

	K_a	K_o	K_p	γ (kN/m^3)
Engineered Fill	0.38	0.58	2.66	22.0
Structural Fill	0.31	0.47	3.30	23.0
Native Soils	0.38	0.55	2.66	22.5

4.16 PERMANENT LATERAL WALL PRESSURES

The distribution of soil pressure against a permanent wall may be assumed using the general equation given above with a coefficient of lateral earth pressure equal to the at rest coefficient of earth pressure, $k = k_o$. Values of k_o are given above for fill and native silt and clay as permanent walls can be constructed with backfill or poured neat to temporary shoring and native soils.

Permanent walls should be designed to resist the maximum possible water pressure subject to drainage conditions determined by design.

4.17 TEMPORARY LATERAL WALL PRESSURES

The distribution of soil pressure against a temporary wall may be assumed using the general equation given above and values of K according to deformation restrictions as follows:

- If moderate wall movements can be permitted
  $K=K_a$.
- If foundations of buildings or services exist at a shallow depth, at a distance less than H (height of the wall) behind the top of the wall and not closer than 0.5H
  $K= 0.5 (K_a + K_o)$.
- If foundations or services exist at a shallow depth, at a distance less than 0.5H
  $K=K_o$.

4.18 TEMPORARY PASSIVE WALL RESISTANCE

Passive resistance at the base of a temporary wall may be calculated as follows:

$$P'_p = K_p (\gamma' d/1.5)$$

Where P'_p = passive resistance at depth below excavation (kN/m^2)
K_p = coefficient of passive earth pressure
γ' = submerged unit weight (kN/m^3)
d = depth below excavation level (m)

4.19 BACKFILL MATERIALS AND COMPACTION

The on-site materials may be suitable for use as general engineered or structural fill subject to material evaluation and removal of deleterious materials. Imported fill should be approved for use as structural or general engineered fill.

Recommended compaction specifications and materials are as follows:

- Structural fill - 100 percent Standard Proctor maximum dry density, maximum compacted lift thickness 250 mm, maximum grain size 200 mm. Structural fill materials should comprise clean well-graded inorganic granular soils.
- General engineered fill - 98 percent Standard Proctor maximum dry density, 0 to +3 percent of optimum moisture content, maximum compacted lift thickness 300 mm. General engineered fill

materials should comprise clean well-graded granular soils, or inorganic medium to low plastic cohesive soils.

Where washing of fines is possible, fill material placed should be separated from coarser or finer material by a suitable geotextile.

Backfill comprising cohesive soils should be considered frost susceptible and should not be used in areas where it may become frozen and where frost heaving would be unacceptable.

5.0 REVIEW OF DESIGN AND CONSTRUCTION

M•L should review details of the design and specifications related to geotechnical aspects prior to construction. Adequate monitoring during construction will be required. All construction should be carried out by a qualified contractor experienced in foundation and earthworks construction. Adequate monitoring includes:

- Shallow Foundations - Inspection by a qualified geotechnical engineer prior to placement of footings.
- Earthworks - Full-time monitoring and compaction testing.
- Deep Utility Installation – Full-time monitoring and compaction testing.

All monitoring should be carried out by a qualified person, independent of the contractor. M•L will provide these services if requested. Failure to provide an adequate level of foundation monitoring may be contravention of building code requirements.

5.1 DESIGN AND CONSTRUCTION GUIDELINES

Recommended general design and construction guidelines are provided in Appendix B under the following headings:

- Backfill Materials and Compaction
- Proof-Rolling
- Construction Excavations
- Floor Slabs-On-Grade
- Shallow Foundations
- Bored Cast-in-Place Concrete Piles

These guidelines are intended to present standards of good practice. Although supplemental to the main text of this report, they should be interpreted as part of the report. Design recommendations presented herein are based on the premise that these guidelines will be followed. The design and construction guidelines are not intended to represent detailed specifications for the work, although they prove useful in the preparation of such specifications. In the event of any discrepancy between the main text of this report and Appendix B, the main text should govern.

6.0 LIMITATIONS

Recommendations presented herein are based on a geotechnical evaluation of the findings in eight boreholes. The conditions encountered during the fieldwork are considered to be reasonably representative of the site. If, however, conditions other than those reported are noted during subsequent phases of the project, M•L should be notified and given the opportunity to review our current recommendations in light of new findings. This report does not include any recommendations related to contaminants in soil or groundwater. Should there be any other documentation indicating any excavation or land disturbance, such as environmental reports, M•L would require these reports prior to site development to confirm the recommendations within this report are suitable in light of new information.

This report has been prepared for the exclusive use of GEC Architecture and their agents for specific application to the development described in this report. It has been prepared in accordance with generally accepted soil and foundation engineering practices. No warranty is expressed or implied.

The Town of Drumheller shall at all times be irrevocably and unconditionally entitled to fully rely on this report as an addressee and party to the report, including all attachments, drawings and schedules, in each case notwithstanding any provision, disclaimer or waiver in the report to the contrary.

The Town of Drumheller shall be entitled to provide copies of the report to Town Council and Town of Drumheller employees, Town of Drumheller regulatory boards, affiliates, advisors, consultants, lenders, and assignees, each of whom shall also be similarly entitled to fully rely on the report in their official capacities for the specific purpose for which the report was prepared.

The Town of Drumheller is at all times entitled to provide copies of the report to Alberta Environment and any other governmental authorities and regulatory bodies having jurisdiction. The Town of Drumhller may also contact the author or other parties to the report to obtain further information respecting the report or to discuss the report further.

7.0 CLOSURE

We trust information presented herein meets with your present requirements. If you have questions or require additional geotechnical services please contact our office.

Respectfully submitted,

McIntosh•Lalani Engineering Ltd.

ASAD SHAIKH

July 9, 2019

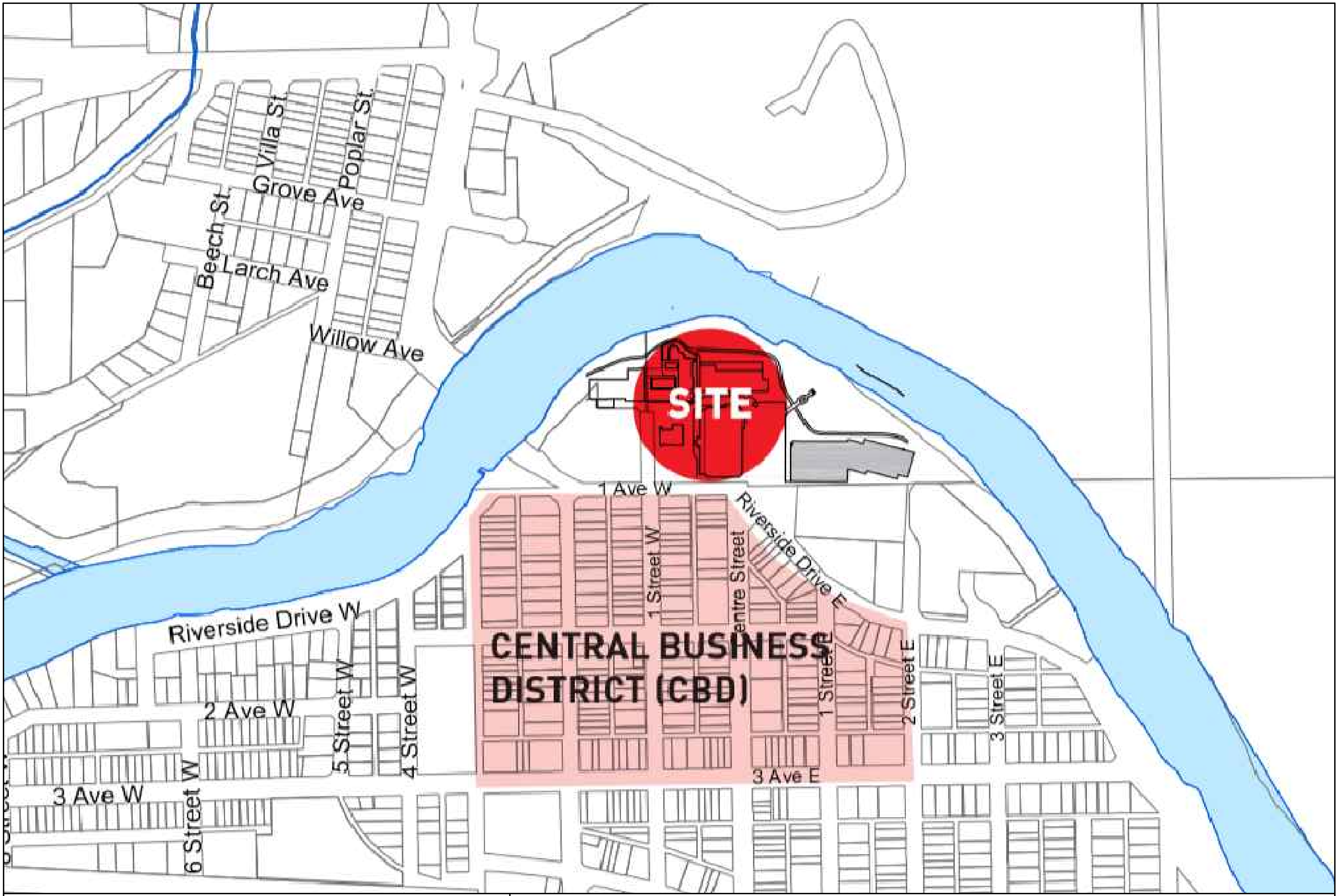
Asad Shaikh, P.Eng.
Senior Project Engineer

MARTIN D. WARD

July 8, 2019

Marty D. Ward, P.Eng.
Director of Engineering
APEGA Permit No. 6482

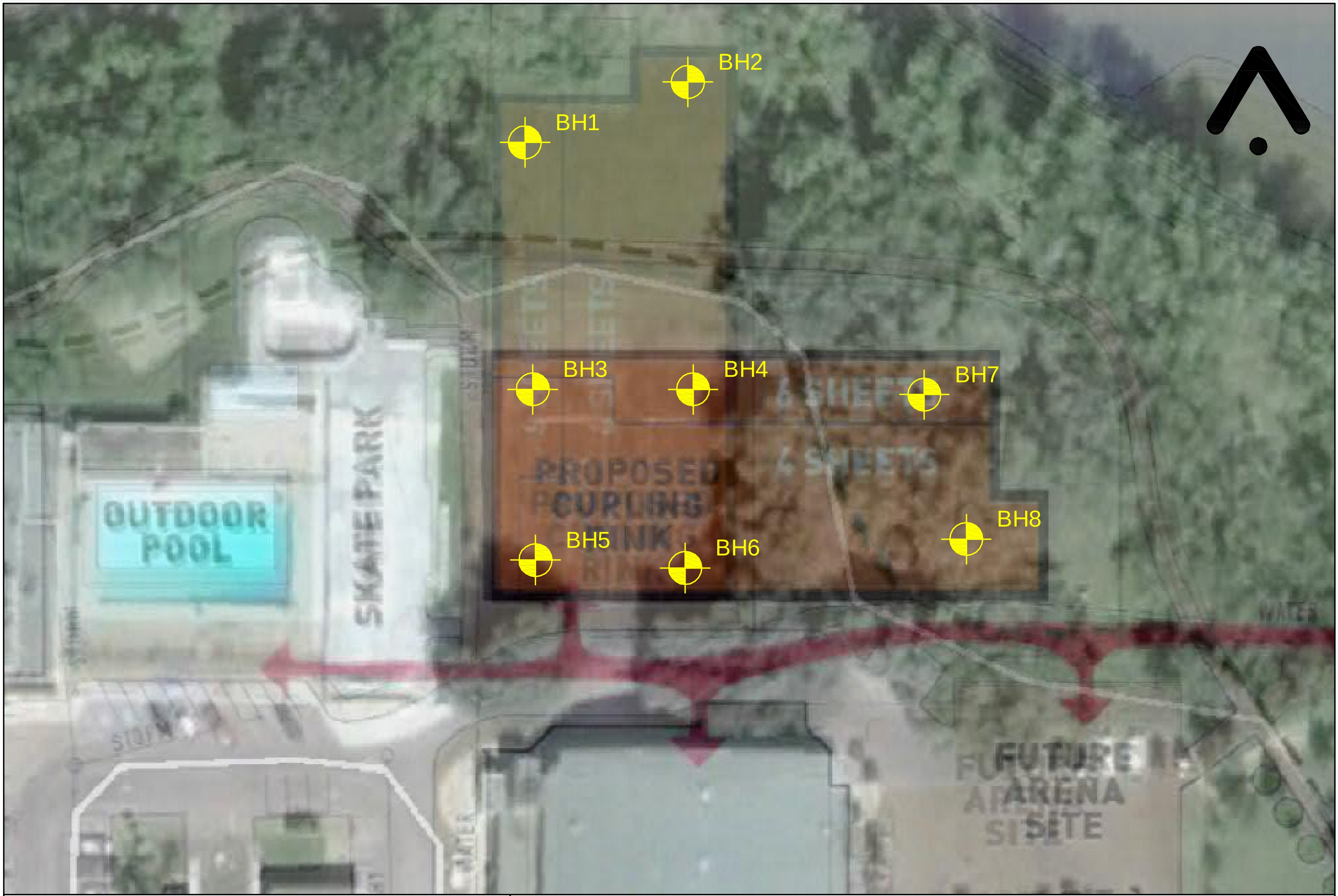
Drawings & Figures



McINTOSH • LALANI ENGINEERING LTD.

Client: GEC Architecture Ltd.
Project: Drumheller Curling Club
Title: Viscinity Map

Job Number:	Drawing Number:	Scale:	Date:
ML-8855	8855.00.G100	N.T.S.	October 11, 2018



Client: GEC Architecture Ltd.
Project: Drumheller Curling Club
Title: Borehole Locations

McINTOSH•LALANI ENGINEERING LTD.

Job Number:	Drawing Number:	Scale:	Date:
ML-8855	8855.00.G101	N.T.S.	October 11, 2018

Appendix A

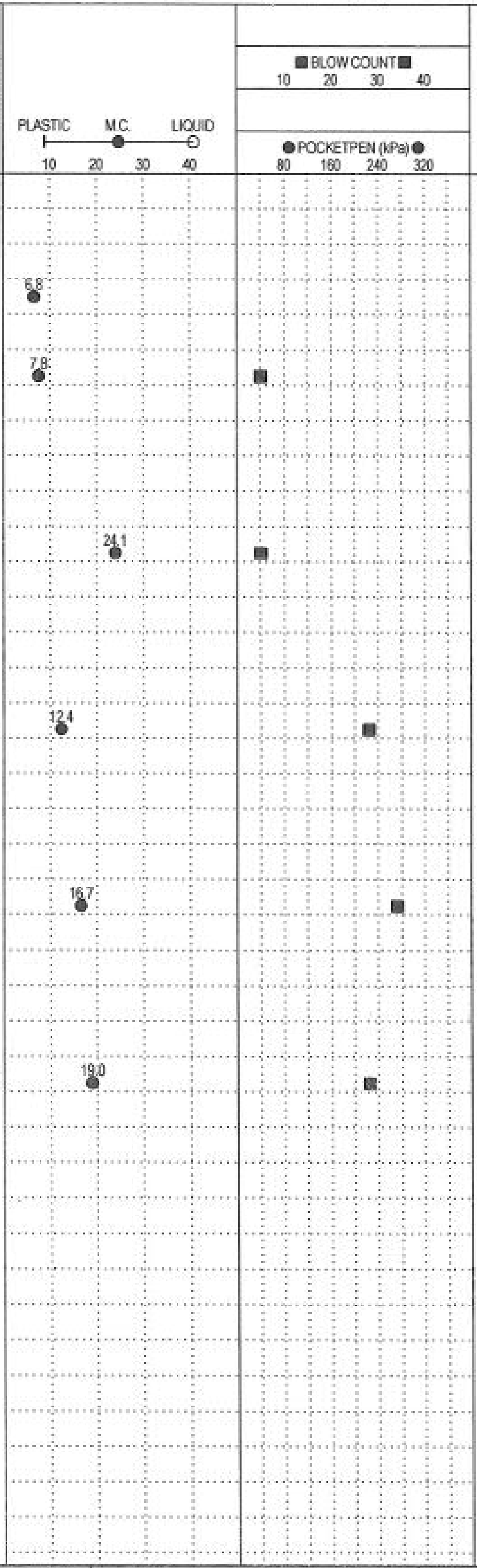
Borehole Logs

Project: Drumheller Curling Club	Drilling Information:	Borehole No.:1
Client: GEC Architecture Ltd.	All Service Drilling Inc	Project No.:ML-8855
	CME 55 track SS-Auger	Elevation:Existing Grade

SAMPLE TYPE ▩ SHELBY TUBE ▮ CORE SAMPLE ☒ SPT SAMPLE GRAB SAMPLE AUGER SAMPLE ▥ NO RECOVERY

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

Depth (m)	SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NO	USCS	BLOWS /150 mm	M.C.
0	TOPSOIL - dark brown organics, trace rubble approx. 450 mm thick.			TPSL		
	Silty SAND - fine grain, loose, damp, trace coal, light to medium brown.	AUGER SAMPLE	1-1			
1		AUGER SAMPLE	1-2			6.8
	- trace roots.	SPT SAMPLE	1-3	SM	3-2-3	7.8
2	- trace clay, moist, trace oxides.	AUGER SAMPLE	1-4			
3		SPT SAMPLE	1-5		2-2-3	24.1
	Sandy GRAVEL - coarse grain, fine gravel, well sorted, compact, moist, medium brown.	AUGER SAMPLE	1-6			
4						
		SPT SAMPLE	1-7		8-16-12	12.4
5	- coarse gravel, wet.	AUGER SAMPLE	1-8			
6		SPT SAMPLE	1-9	GWS	5-14-20	16.7
7		AUGER SAMPLE	1-10			
	- trace organics, med grey.	SPT SAMPLE	1-11		11-15-13	19.0
8		AUGER SAMPLE	1-12			
9	END OF HOLE at a depth of 9.1 m. Slough to a depth of 6.1 m. 25 mm PVC standpipe installed to a depth of 9.1 m with 3.0 m slotted. Wet upon completion. Water Levels: Sept 25, 2018: 4.67 m EOH: 8.80 m					
10						
11						



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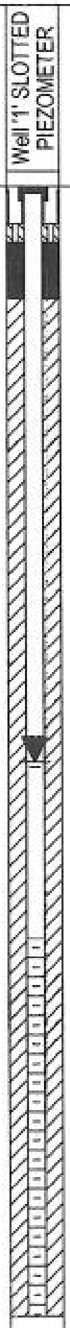


Elevation (m)

McIntosh Lalani Engineering
Calgary, AB
(403) 291-2345

Logged By: RC	Completion Depth: 30 ft
Reviewed By: Asad Shaikh	Drilled on: 2018-09-14
Groundwater Depth: m	Page 1 of 1

Project: Drumheller Curling Club	Drilling Information:	Borehole No.:2
Client: GEC Architecture Ltd.	All Service Drilling Inc	Project No.:ML-8855
	RK-10 SS-Auger	Elevation:Existing Grade

SAMPLE TYPE: SHELBY TUBE · CORE SAMPLE · SPT SAMPLE · GRAB SAMPLE · AUGER SAMPLE · NO RECOVERY

BACKFILL TYPE: BENTONITE · PEA GRAVEL · SLOUGH · GROUT · DRILL CUTTINGS · SAND

Depth (m): 0 – 23

SOIL DESCRIPTION

TOPSOIL - dark brown organics, trace rubble approx. 400 mm thick.

Silty Sand FILL - fine grain, loose, damp, medium brown.
- coal approx. 150 mm thick, trace rubble, trace gravel, trace roots.

trace oxides, trace clay.

Silty SAND - fine grain, loose, wet, trace oxides, trace roots, medium brown.

Sandy GRAVEL - coarse grain, well sorted, wet, medium brown/orange.
- no recovery in split spoon.
- no recovery in split spoon.

Silty CLAY (Till) - trace sand and gravel, soft, wet, low plastic, trace oxide, trace coal, medium brown.

Silty SAND - trace gravel, some clay, loose, wet, medium blue/grey.

Silty CLAY (Till) - trace sand and gravel, stiff, moist-wet, low plastic, trace coal, medium blue/grey.
- medium to high plastic, damp, medium grey.
- no recovery in split spoon.

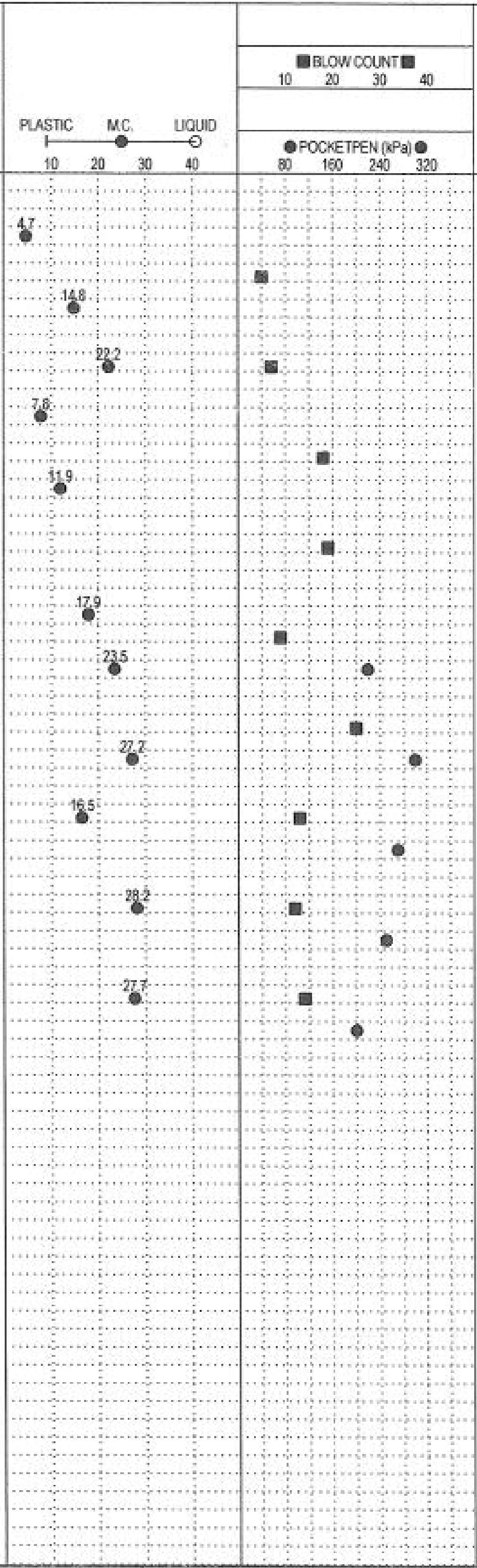
END OF HOLE at a depth of 15.2 m. Slough to a depth of 11.8 m. 25 mm PVC standpipe installed to a depth of 14.4 m with 1.5 m slotted. Wet upon completion.

WELL B: 25 mm PVC standpipe installed, (nested with 1 in same well) to a depth of 7.6 m with 1.5 m slotted. Wet upon completion.

Water Levels:
Sept 25, 2018: 11.58 m EOH: 12.78 m

Well B:
Sept 25, 2018: 6.12 m EOH: 7.57 m

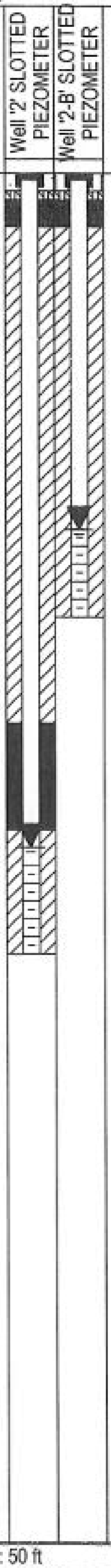
SAMPLE TYPE	SAMPLE NO	USCS	BLOWS /150 mm
AUGER SAMPLE	2-1	TPSL	
AUGER SAMPLE	2-2		
SPT SAMPLE	2-3		3-3-2
AUGER SAMPLE	2-4	FILL	
SPT SAMPLE	2-5		2-3-4
AUGER SAMPLE	2-6	SM	
AUGER SAMPLE	2-7		
SPT SAMPLE	-		6-9-9
AUGER SAMPLE	2-8	GWS	
SPT SAMPLE	-		7-10-9
AUGER SAMPLE	2-9	CL-ML	
AUGER SAMPLE	2-10	SM	
SPT SAMPLE	2-11		3-3-6
AUGER SAMPLE	2-12		
AUGER SAMPLE	2-13		
SPT SAMPLE	-		9-10-15
AUGER SAMPLE	2-14		
SPT SAMPLE	2-15		3-6-7
AUGER SAMPLE	2-16	CL-ML	
SPT SAMPLE	2-17		4-5-7
AUGER SAMPLE	2-18		
SPT SAMPLE	2-19		3-6-8
AUGER SAMPLE	2-20		



OTHER DATA

Organic Content = 2.6%

[SO$_4$] = 0.041%



Elevation (m)

McIntosh Lalani Engineering
Calgary, AB
(403) 291-2345

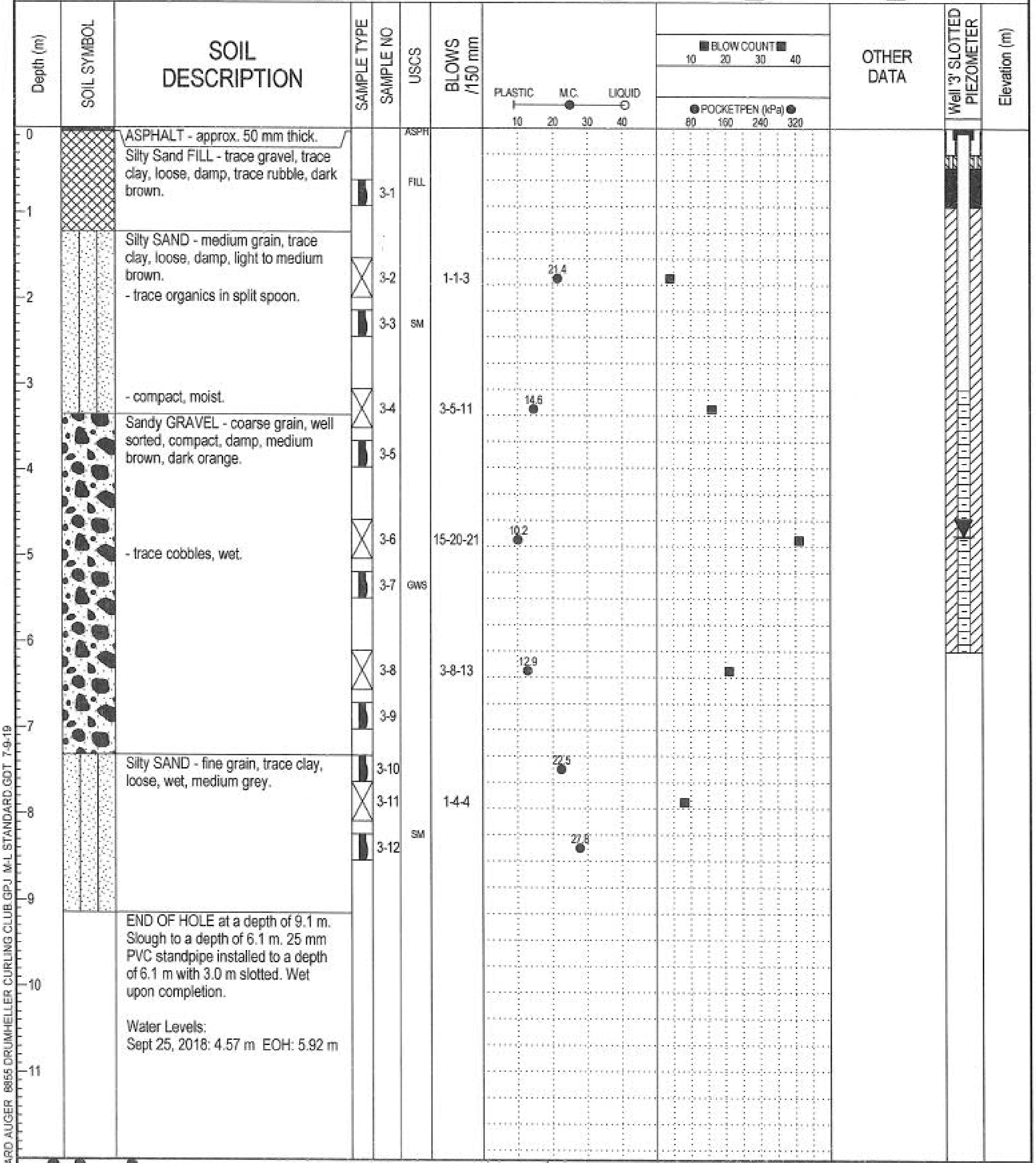
Logged By: RC	Completion Depth: 50 ft
Reviewed By: Asad Shaikh	Drilled on: 2018-09-13
Groundwater Depth: m	Page 1 of 1

ML STANDARD AUGER 8855 DRUMHELLER CURLING CLUB.GPJ M-L STANDARD.GDT 7-9-19

Project: Drumheller Curling Club	Drilling Information:	Borehole No.:3
Client: GEC Architecture Ltd.	All Service Drilling Inc	Project No.:ML-8855
	CME 55 track SS-Auger	Elevation:Existing Grade

SAMPLE TYPE: ■ SHELBY TUBE ▯ CORE SAMPLE ☒ SPT SAMPLE GRAB SAMPLE AUGER SAMPLE ▥ NO RECOVERY

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



SOIL DESCRIPTION

ASPHALT - approx. 50 mm thick.

Silty Sand FILL - trace gravel, trace clay, loose, damp, trace rubble, dark brown.

Silty SAND - medium grain, trace clay, loose, damp, light to medium brown.
- trace organics in split spoon.
- compact, moist.

Sandy GRAVEL - coarse grain, well sorted, compact, damp, medium brown, dark orange.
- trace cobbles, wet.

Silty SAND - fine grain, trace clay, loose, wet, medium grey.

END OF HOLE at a depth of 9.1 m. Slough to a depth of 6.1 m. 25 mm PVC standpipe installed to a depth of 6.1 m with 3.0 m slotted. Wet upon completion.

Water Levels:
Sept 25, 2018: 4.57 m EOH: 5.92 m

SAMPLE NO	USCS	BLOWS /150 mm	M.C.
	ASPH		
3-1	FILL		
3-2		1-1-3	21.4
3-3	SM		
3-4		3-5-11	14.6
3-5			
3-6		15-20-21	10.2
3-7	GWS		
3-8		3-8-13	12.9
3-9			
3-10			22.5
3-11		1-4-4	
3-12	SM		27.8

McIntosh Lalani Engineering
Calgary, AB
(403) 291-2345

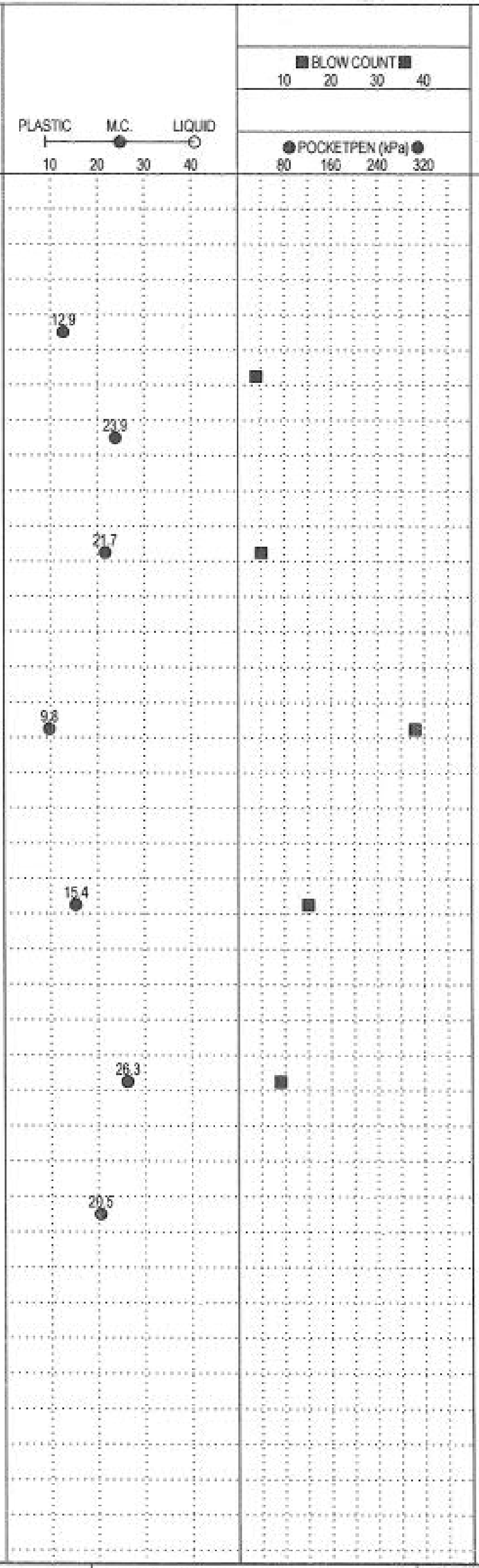
Logged By: RC	Completion Depth: 30 ft
Reviewed By: Asad Shaikh	Drilled on: 2018-09-14
Groundwater Depth: m	Page 1 of 1

Project: Drumheller Curling Club	Drilling Information:	Borehole No.:4
Client: GEC Architecture Ltd.	All Service Drilling Inc	Project No.:ML-8855
	CME 55 track SS-Auger	Elevation:Existing Grade

SAMPLE TYPE ▩ SHELBY TUBE ▮ CORE SAMPLE ☒ SPT SAMPLE ▣ GRAB SAMPLE ▮ AUGER SAMPLE ▥ NO RECOVERY

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

Depth (m)	SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NO	USCS	BLOWS /150 mm	M.C.
0	ASPHALT - approx. 50 mm thick.			ASPH		
	Silty Sand FILL - trace gravel, trace clay, loose, damp, trace rubble, dark brown.	▮	4-1	FILL		
1	Silty SAND - fine grain, loose, damp, trace roots, light brown.	▮	4-2	SM		12.9
		☒	4-3		2-2-2	
2	- trace clay, moist.	▮	4-4			23.9
3		☒	4-5		3-2-3	21.7
4	Sandy GRAVEL - coarse grain, well sorted, compact, damp, medium brown, dark orange.	▮	4-6	GWS		
		☒	4-7		11-20-18	9.8
5	- wet, trace cobbles.	▮	4-8			
6		☒	4-9		2-4-11	15.4
7	Silty SAND - fine grain, trace gravel, loose, wet, medium grey/green.	▮	4-10	SM		
8		☒	4-11		3-4-5	26.3
9	- trace organics.	▮	4-12			20.5



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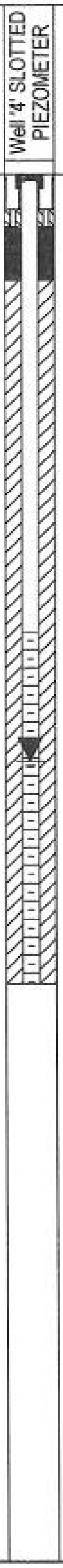


Elevation (m)

END OF HOLE at a depth of 9.1 m. Slough to a depth of 6.1 m. 25 mm PVC standpipe installed to a depth of 7.0 m with 3.0 m slotted. Wet upon completion.

Water Levels:
Sept 25, 2018: 5.08 m EOH: 6.92 m

ML STANDARD AUGER 8855 DRUMHELLER CURLING CLUB.GPJ M-L STANDARD.GDT 7-9-19

McIntosh Lalani Engineering
Calgary, AB
(403) 291-2345

Logged By: RC	Completion Depth: 30 ft
Reviewed By: Asad Shaikh	Drilled on: 2018-09-14
Groundwater Depth: m	Page 1 of 1

Project: Drumheller Curling Club	Drilling Information:	Borehole No.:5
Client: GEC Architecture Ltd.	All Service Drilling Inc	Project No.:ML-8855
	CME 55 track SS-Auger	Elevation:Existing Grade

SAMPLE TYPE ▇ SHELBY TUBE ▯ CORE SAMPLE ☒ SPT SAMPLE ▨ GRAB SAMPLE ▙ AUGER SAMPLE ▥ NO RECOVERY

BACKFILL TYPE ▇ BENTONITE ⁘ PEA GRAVEL ▥ SLOUGH ⁘ GROUT ▧ DRILL CUTTINGS ⁘ SAND

Depth (m) | SOIL SYMBOL | SOIL DESCRIPTION | SAMPLE TYPE | SAMPLE NO | USCS | BLOWS /150 mm | OTHER DATA | Well '5' SLOTTED PIEZOMETER | Elevation (m)

ASPHALT - approx. 50 mm thick.

Silty Sand FILL - trace gravel, trace clay, loose, damp, trace rubble, dark brown.

Silty SAND - fine grain, loose, damp, trace roots, light brown.

- trace clay, moist.

Sandy GRAVEL - coarse grain, well sorted, compact, damp, medium brown, dark orange.

- wet, trace cobbles, trace organics.

Silty SAND - fine grain, some clay, loose, wet, medium grey.

Sandy GRAVEL - coarse grain, well sorted, compact, damp, trace cobbles, medium brown, dark orange.

- no recovery in split spoon, no recovery on auger.

- no recovery on auger.

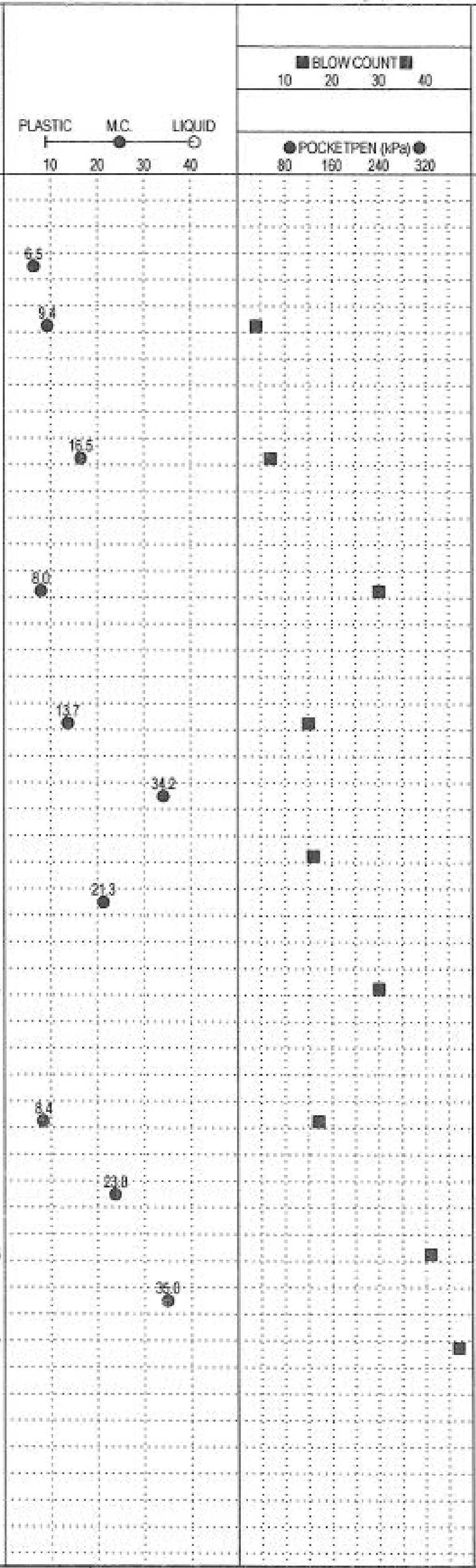
Shale BEDROCK - weathered, weak, damp, light grey.

Coal BEDROCK - weathered weak, wet, black.

END OF HOLE at a depth of 13.7 m. Slough to a depth of 6.1 m. 25 mm PVC standpipe installed to a depth of 8.2 m with 3.0 m slotted. Wet upon completion.

Water Levels:
Sept 25, 2018: 5.46 m EOH: 8.25 m

		ASPH			
▙	5-1	FILL			
▙	5-2			6.5	Organic Content = 2.4%
☒	5-3		2-2-2	9.4	
		SM			
▙	5-4				
☒	5-5		3-3-4	16.5	
▙	5-6				
☒	5-7		8-15-15	8.0	[SO$_4$] = 0.082%
▙	5-8	GWS			
☒	5-9		7-7-8	13.7	
▙	5-10	SM		34.2	
☒	-		5-6-10		
▙	5-11			21.3	
☒	5-12	GWS	20-15-15		
☒	5-13		6-7-10	8.4	
▙	5-14				
▙	5-15			23.8	
		BE			
☒	5-16		24-25-16		
▙	5-17			35.0	
		BE			
☒	5-18		14-20-27		



ML STANDARD AUGER 8855 DRUMHELLER CURLING CLUB.GPJ M-L STANDARD.GDT 7-8-19

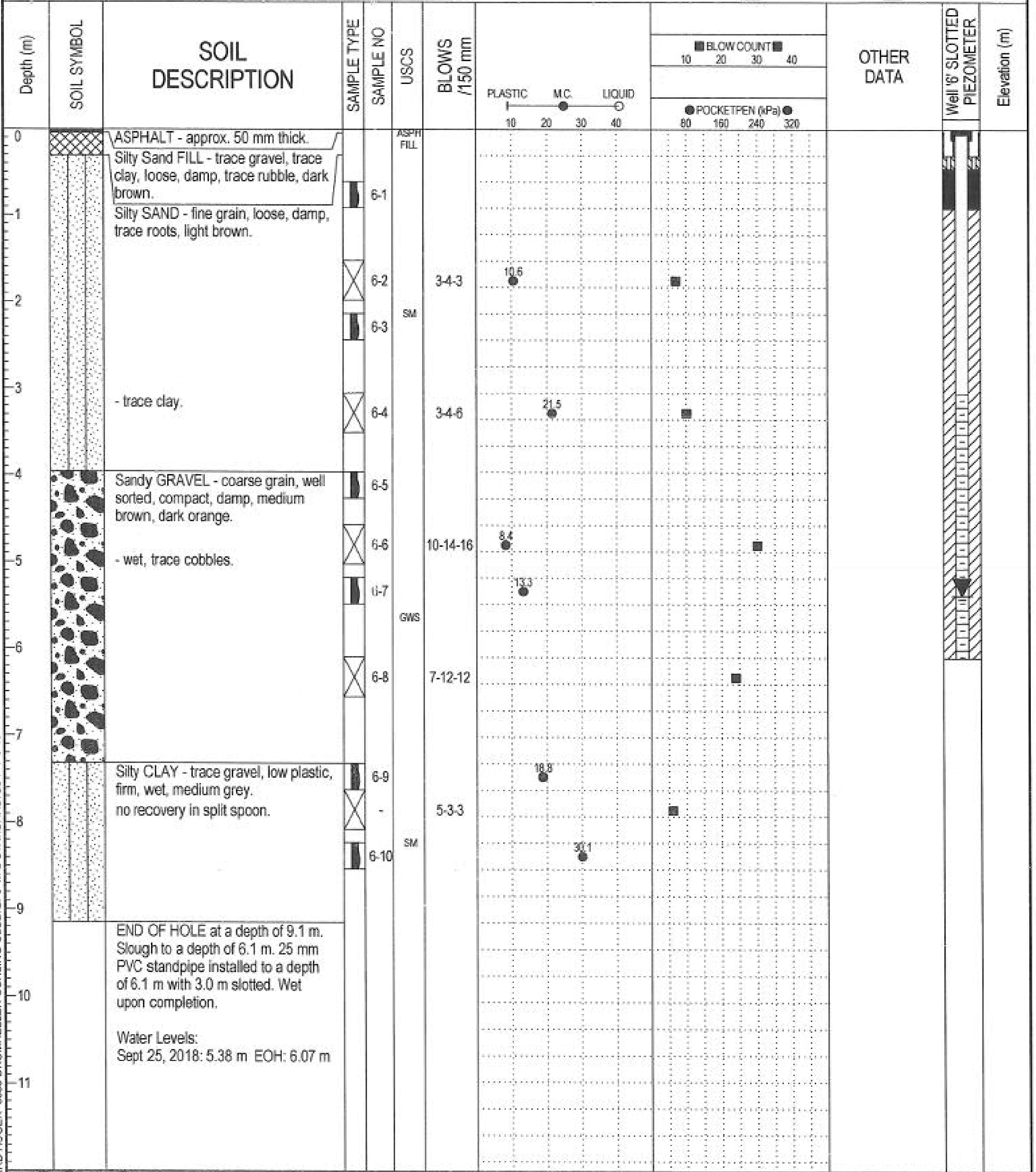
McIntosh Lalani Engineering
Calgary, AB
(403) 291-2345

Logged By: RC	Completion Depth: 45 ft
Reviewed By: Asad Shaikh	Drilled on: 2018-09-14
Groundwater Depth: m	Page 1 of 1

Project: Drumheller Curling Club	Drilling Information:	Borehole No.:6
Client: GEC Architecture Ltd.	All Service Drilling Inc	Project No.:ML-8855
	CME 55 track SS-Auger	Elevation:Existing Grade

SAMPLE TYPE ▨ SHELBY TUBE ▯ CORE SAMPLE ☒ SPT SAMPLE ▨ GRAB SAMPLE ▯ AUGER SAMPLE ▥ NO RECOVERY

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



SOIL DESCRIPTION

ASPHALT - approx. 50 mm thick.

Silty Sand FILL - trace gravel, trace clay, loose, damp, trace rubble, dark brown.

Silty SAND - fine grain, loose, damp, trace roots, light brown.

- trace clay.

Sandy GRAVEL - coarse grain, well sorted, compact, damp, medium brown, dark orange.

- wet, trace cobbles.

Silty CLAY - trace gravel, low plastic, firm, wet, medium grey.

no recovery in split spoon.

END OF HOLE at a depth of 9.1 m. Slough to a depth of 6.1 m. 25 mm PVC standpipe installed to a depth of 6.1 m with 3.0 m slotted. Wet upon completion.

Water Levels:
Sept 25, 2018: 5.38 m EOH: 6.07 m

SAMPLE NO	USCS	BLOWS /150 mm	M.C.
	ASPH FILL		
6-1			
6-2	SM	3-4-3	10.6
6-3			
6-4		3-4-6	21.5
6-5			
6-6		10-14-16	8.4
6-7	GWS		13.3
6-8		7-12-12	
6-9			18.8
-		5-3-3	
6-10	SM		30.1

ML STANDARD AUGER 8855 DRUMHELLER CURLING CLUB.GPJ ML STANDARD.GDT 7-8-19

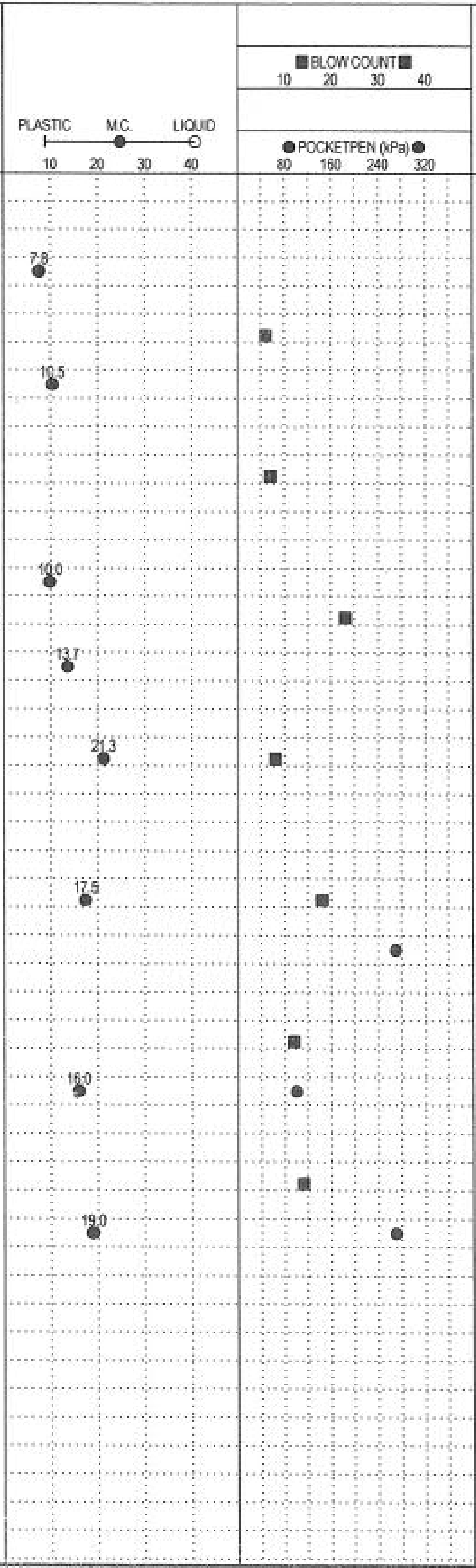
McIntosh Lalani Engineering Calgary, AB (403) 291-2345	Logged By: RC	Completion Depth: 30 ft
	Reviewed By: Asad Shaikh	Drilled on: 2018-09-14
	Groundwater Depth: m	Page 1 of 1

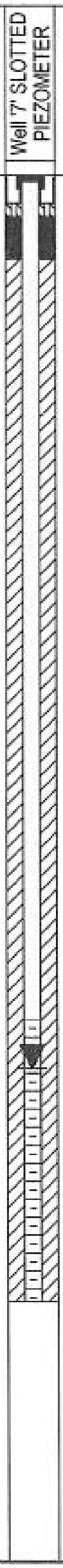
Project: Drumheller Curling Club	Drilling Information:	Borehole No.:7
Client: GEC Architecture Ltd.	All Service Drilling Inc	Project No.:ML-8855
	RK-10 SS-Auger	Elevation:Existing Grade

SAMPLE TYPE ■ SHELBY TUBE ▯ CORE SAMPLE ☒ SPT SAMPLE ▨ GRAB SAMPLE ▮ AUGER SAMPLE ▥ NO RECOVERY

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

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NO	USCS	BLOWS /150 mm	OTHER DATA
TOPSOIL - dark brown organics, trace rubble approx. 450 mm thick. over clayey silt loam	▮	7-1	TPSL		
Silty SAND - trace gravel, loose, damp, trace roots, light brown.	▮	7-2	SM		
	☒	7-3		2-3-3	
	▮	7-4			[SO$_4$] = 0.041%
- trace coal.					
	☒	7-5		3-3-4	
Sandy GRAVEL - coarse grain, well sorted, compact, wet, trace cobbles, medium brown, dark orange.	▮	7-6	GWS		
	☒	7-7		5-10-13	
	▮	7-8			
Silty CLAY - trace sand and gravel, soft, moist, low plastic, medium brown.	☒	7-9	CL-ML	3-4-4	
Sandy GRAVEL - some clay, coarse, compact, medium brown.	▮	7-10	GWS		
Silty CLAY - trace sand and coarse gravel, stiff, moist, low plastic, medium grey.	☒	7-11	CL-ML	9-8-10	
	▮	7-12			
- no recovery in split spoon.	☒	-		5-6-6	
	▮	7-13			
- no recovery in split spoon.	☒	-		4-6-8	
	▮	7-14			
END OF HOLE at a depth of 12.2 m. Slough to a depth of 10.6 m. 25 mm PVC standpipe installed to a depth of 12.2 m with 3.0 m slotted. Wet upon completion. Water Levels: Sept 25, 2018: 9.68 m EOH: 11.82 m					





McIntosh Lalani Engineering
Calgary, AB
(403) 291-2345

Logged By: RC	Completion Depth: 40 ft
Reviewed By: Asad Shaikh	Drilled on: 2018-09-13
Groundwater Depth: m	Page 1 of 1

Project: Drumheller Curling Club	Drilling Information:	Borehole No.:8
Client: GEC Architecture Ltd.	All Service Drilling Inc	Project No.:ML-8855
	CME 55 track SS-Auger	Elevation:Existing Grade

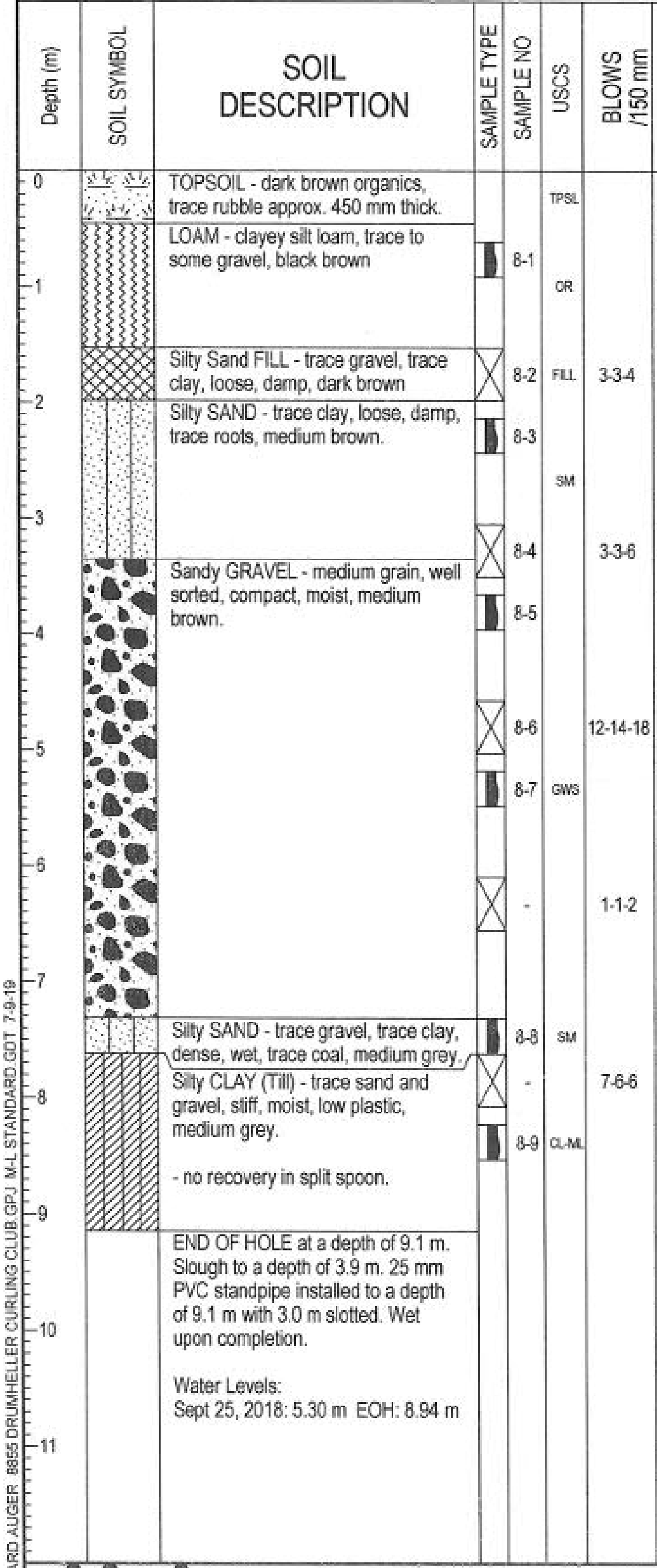
SAMPLE TYPE ■ SHELBY TUBE ▮ CORE SAMPLE ☒ SPT SAMPLE ▨ GRAB SAMPLE ▮ AUGER SAMPLE ▯ NO RECOVERY

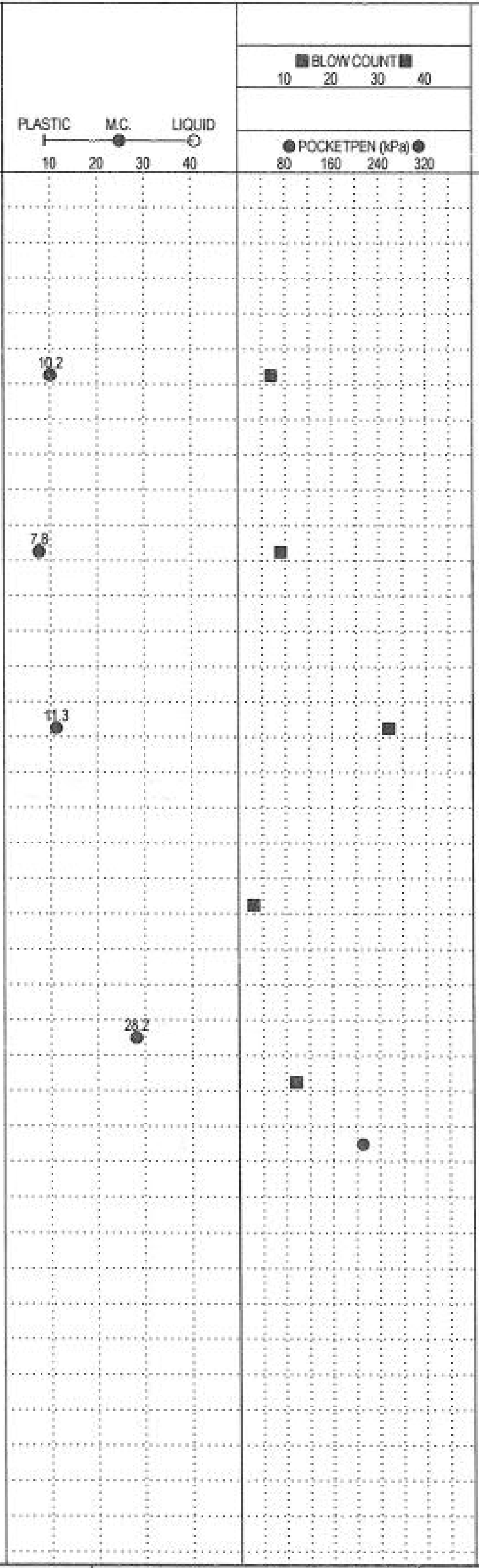
BACKFILL TYPE ■ BENTONITE ⁚ PEA GRAVEL ▥ SLOUGH ▨ GROUT ▨ DRILL CUTTINGS ⁚ SAND

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NO	USCS	BLOWS /150 mm	M.C.
TOPSOIL - dark brown organics, trace rubble approx. 450 mm thick.			TPSL		
LOAM - clayey silt loam, trace to some gravel, black brown	▮	8-1	OR		
Silty Sand FILL - trace gravel, trace clay, loose, damp, dark brown	☒	8-2	FILL	3-3-4	10.2
Silty SAND - trace clay, loose, damp, trace roots, medium brown.	▮	8-3	SM		
	☒	8-4		3-3-6	7.8
Sandy GRAVEL - medium grain, well sorted, compact, moist, medium brown.	▮	8-5			
	☒	8-6		12-14-18	11.3
	▮	8-7	GWS		
	☒	-		1-1-2	
Silty SAND - trace gravel, trace clay, dense, wet, trace coal, medium grey.	▮	8-8	SM		28.2
Silty CLAY (Till) - trace sand and gravel, stiff, moist, low plastic, medium grey. - no recovery in split spoon.	☒	-		7-6-6	
	▮	8-9	CL-ML		

END OF HOLE at a depth of 9.1 m. Slough to a depth of 3.9 m. 25 mm PVC standpipe installed to a depth of 9.1 m with 3.0 m slotted. Wet upon completion.

Water Levels:
Sept 25, 2018: 5.30 m EOH: 8.94 m





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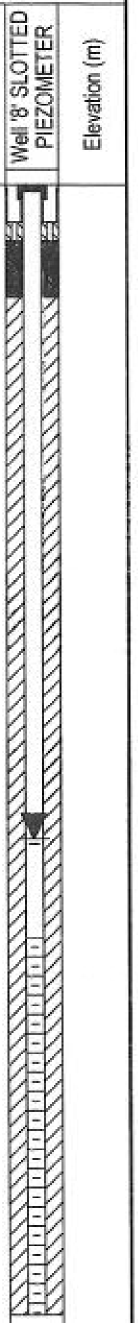

McIntosh Lalani Engineering
Calgary, AB
(403) 291-2345

Logged By: RC	Completion Depth: 30 ft
Reviewed By: Asad Shaikh	Drilled on: 2018-09-14
Groundwater Depth: m	Page 1 of 1

Appendix B

Design & Construction Guidelines

B.1 BACKFILL MATERIALS AND COMPACTION

Maximum density, as used in this section, means Standard Proctor Maximum Dry Density (ASTM Test D698) unless otherwise noted. Optimum moisture content is as defined in this text.

Backfill adjacent to exterior footings, foundation walls, grade beams and pile caps and within 300 mm of final grade should comprise low-plastic cohesive general engineered fill as defined above. Such backfill should provide a relatively impervious surface layer to reduce seepage in the sub-soil.

Backfill should not be placed against a foundation structure until the structure has sufficient strength to withstand the earth pressures resulting from placement and compaction. During compaction, careful observation of the foundation wall for deflection should be carried out continuously. Where deflection is apparent, the compactive effort should be reduced accordingly. In order to reduce potential compaction induced stresses, only hand held compaction equipment should be used in the compaction of fill within 500 mm of retaining walls or basement walls.

Backfill materials should not be placed in a frozen state or placed on a frozen subgrade. All lumps of materials should be broken down during placement.

Where the maximum-sized particles in any backfill material exceed 50 percent of the lift thickness or minimum dimension of the cross-section to be backfilled, such particles should be removed and placed at other more suitable locations on site or screened-off prior to delivery to site.

Bonding should be provided between backfill lifts, if the previous lift has become desiccated. For fine-grained materials, the previous lift should be scarified to 75 mm in depth followed by proper moisture conditioning and recompaction.

B.1.1 GENERAL ENGINEERED FILL

Backfill adjacent to and above footings, abutment walls, basement walls, grade beams and pile caps or below highway, street or parking lot pavement sections should comprise general engineered fill. “General engineered fill” materials should comprise clean, well-graded granular soils or inorganic, low-plastic cohesive soils. Such material should be placed in lifts not exceeding an uncompacted thickness of 300 mm, and compacted to not less than 98 percent of maximum density, at a moisture content at or slightly above optimum. The uncompacted lift thickness may be adjusted based on the method of fill placement and the size and type of compaction equipment in use.

B.1.2 STRUCTURAL FILL

Backfill supporting structural loads should comprise structural fill materials. “Structural fill” materials should comprise clean, well-graded inorganic granular soils. Such fill should be placed in compacted lifts not exceeding 150 mm and compacted to not less than 98 percent of maximum density, at a moisture content at or slightly (0 to 3 percent) above optimum. The following table provides gradation limits for structural fill of various nominal sizes. The gradation limits have been adapted from the City of Calgary Roads Construction 2015 Standard Specifications, Section 303.00.00 Materials. Other gradations may be approved on a project specific basis by a qualified geotechnical engineer.

Sieve Size (mm)	**Percent Passing By Weight** **Nominal Gravel Size**		
	80 mm	**50 mm**	**25 mm**
80	100		
75	---		
50	---	100	
40	60 - 90	95 – 100	
25	---	---	100
20	40 - 70	50 – 75	95 - 100
10	25 - 60	25 - 52	55 - 80
5	15 - 45	15 - 40	35 - 65
2.5	10 - 35	10 - 33	28 - 52
0.63	5 - 23	5 - 23	13 - 35
0.315	---	---	9 - 26
0.16	3 - 12	2 - 14	6 - 18
0.08	2 - 10	1 - 10	4 - 10
%Fractures (2 faces)	20	30	60

B.1.3 LEAN MIX CONCRETE

"Lean-mix concrete" should be low strength concrete having a minimum 28 days compressive strength of 3.5 MPa.

B.1.4 LANDSCAPE FILL

"Landscape fill" material may comprise soils without regard to engineering quality. Such soils should be placed in compacted lifts not exceeding 300 mm and compacted to a density of not less than 90 percent of maximum density.

B.1.5 PIPE BEDDING AND DRAINAGE

Bedding for pipes and utilities should generally conform to the manufacturer's specification. The type and depth of bedding material relative to the size of pipe are a function of the rigidity of the utility and the embedment depth. For drainage blankets and weeping tile, an open-graded, clean aggregate is required. The following table represents the gradation limits for bedding gravel. The gradation limits have been adapted from the City of Calgary Standard Specifications: Sewer Construction 2012 Section 402.10.00. Class IA material as defined in the table is also suitable for use in drainage applications. Local municipal specifications or manufacturer's specifications may be substituted at the discretion of a qualified engineer.

Sieve Size (mm)	For Pipe 375 mm and Smaller (20 mm Nominal Size) % passing by mass		Sieve Size (mm)	For Pipe Larger than 375 mm (40 mm Nominal Size) % passing by mass
Class IA*				
20	100		40	100
4.75	0 - 10		4.75	0 – 10
2.5	0 – 5		2.5	0 – 5
0.075	0 - 5		0.075	0 – 5
Class IB				
20	100		40	100
4.75	10 – 50		4.75	10 – 50
2.5	0 – 5		2.5	0 – 5
0.075	0 – 5		0.075	0 – 5
Class II				
20	100		40	100
4.75	0 – 100		4.75	0 – 100
0.075	0 – 12		0.075	0 – 12
Class III				
20	100		40	100
4.75	0 – 100		4.75	0 – 100
0.075	12 – 50		0.075	12 – 50

* Class IA material is suitable for granular material below slabs-on-grade for which a subfloor depressurization system is required for soil gas control, as specified in section 9.16.2.1 of the 2014 Alberta Building Code Volume 2.

B.2 BORED CAST-IN-PLACE CONCRETE PILES

Design and construction of piles should comply with relevant Building Code requirements.

Piles should be installed under full-time inspection of geotechnical personnel. Pile design parameters should be reviewed in light of the findings of the initial bored shafts drilled on a site. Further design review may be necessary if conditions observed during site construction do not conform to design assumptions.

Where fill material, lenses or strata of sand, silt or gravel are present within the designed pile depth, these may be incompetent and/or water bearing and may cause sloughing. Casing should be on hand before drilling starts and be used, if necessary, to seal water and/or prevent sloughing of the hole.

If piles are to be under-reamed (belled), the under-reams should be formed entirely in self-supporting soil and entirely within the competent bearing stratum. Where caving occurs at design elevation, it may be necessary to extend the base of the pile bell to a greater depth. Piles may be constructed with bell having outside diameters up to approximately three times the diameters of their shafts. Piles with shaft diameters of less than 760 mm should not be under-reamed due to difficulties associated with ensuring a clean base.

Prior to pouring concrete, bottoms of pile bells or of straight-shaft end-bearing piles should be cleaned of all disturbed material.

Pile excavation should be visually inspected after completion to ensure that disturbed materials and/or water are not present on the base so that recommended allowable bearing and skin friction parameters may apply.

Visual inspection may be accomplished by the inspector descending into the pile shaft [shaft diameter of 760 mm (30 inches) or greater]. A protective cage and other safety equipment required by government regulations should be provided by the contractor to facilitate down hole inspection.

Other procedures to inspect the pile shafts may be used where shaft diameters of less than 760 mm (30 inch) are constructed, such as inspection with a light.

For safety reasons, where hand cleaning and/or “down shaft” inspection by personnel are required, the pile shaft should be cased full-length prior to personnel entering the shaft.

Reinforcing steel should be on hand and should be placed as soon as the bore has been completed and approved.

Longitudinal reinforcing steel is recommended to counteract the possible tensile stresses induced by frost action and should extend to a minimum depth of 3.5 m. A minimum steel of 0.5 percent of the gross shaft area is recommended.

Where a limited quantity of water is present on the pile base, when permitted or directed by a geotechnical engineer, it should be either removed or absorbed by the addition of dry cement, which should then be thoroughly mixed as an in situ slurry by means of the belling tool, using reverse rotation of the tool. Where significant quantities of water are present and it is impracticable to exclude water from the pile bore, concrete should be placed by tremie techniques or concrete pump.

A “dry” pile should be poured by “free fall” of concrete only where impact of the concrete against the reinforcing cage, which can cause segregation of the concrete, will not occur. A hopper should be used to direct concrete down the centre of the pile base and to prevent impact of concrete against reinforcing steel.

Concrete used for dry piles should be self-compacting and should have a slump of between 50 mm and 130 mm. Concrete for each pile should be poured in one continuous operation and should be placed immediately after excavation and inspection of piles, to reduce the opportunity for the ingress of free water or deterioration of the exposed soil or rock.

If piles cannot be formed in dry conditions, then the concrete should be placed by tremie tube or concrete pump. Concrete placed by tremie should have a slump of not less than 150 mm. A ball or float should be used in the tremie tube to separate the initial charge of concrete from the water in the pile hole.

The outlet of the tremie tube should be maintained at all times 1.0 m to 2.0 m below the surface of the concrete. The diameter of the tremie tube should be at least 200 mm. The tube should be water-tight and not be made of aluminum. Smaller diameter pipes may be used with a concrete pump. The surface of the concrete should be allowed to rise above the cut-off level of the pile, so that when the temporary casing is withdrawn and the surface level of the concrete adjusts to the new volume, the top of the uncontaminated concrete is at or above the cut-off level. The concrete should be placed in one continuous, smooth operation without any halts or delays. Placing the lower portion of the pile by tremie tube and placing the upper portion of the pile by free fall should not be permitted, to ensure that defects in the pile shaft at the top of the tremie concrete do not occur.

As the surface of the concrete rises in the pile bore, the water in the pile bore will be displaced upwards and out of the top of the pile casing. It may be necessary to pump off this water to a container to temporary ditch drain to prevent the formation of ice or flooding conditions and possibly damage to existing structures.

When concreting by tremie techniques, allowance should be made for the removal of contaminated or otherwise defective concrete at the tops of the piles.

The casing should be filled with concrete and then the casing should be withdrawn smoothly and continuously.

Sufficient concrete should be placed to allow for additional volume of the casing and reduction in level of the concrete as the casing is withdrawn. Concrete should not be poured on top of previously poured concrete after the casing is withdrawn.

An accurate record of the volume of concrete placed should be maintained as a check that a continuous pile has been formed.

Concrete should not be placed if its temperature is less than 5°C or exceeds 30°C or if it is more than two hours old.

Where tension, horizontal or bending moment loading on the pile is foreseen, steel reinforcing should be extended and tied into the grade beam or pile cap. The steel should be designed to transfer loads to the required depth in the pile and to resist resultant bending moments and shear forces.

Void formers should be placed beneath all grade beams to reduce the risk of damage due to frost effects or soil moisture changes.

Where the drilling operation might affect the concrete in adjacent pile (ie. where pile spacing is less than about three diameters), drilling should not be carried out before the previously poured pile concrete has set for at least 24 hours.

Where a group of four or more piles are used, the allowable working load on the piles may need to be modified to allow for group effects.

Piles should be spaced no closer than 2.5 times the pile shaft diameter, measured centre-to-centre. Strict control of pile location and vertically should be exercised to provide accurate locations and spacing of piles. In general, piles should be constructed within a tolerance of 75 mm plan distance in any direction and within a vertically of 1 in 75 mm.

A detailed record should be kept of pile construction including information such as pile number, shaft/base diameter, date and time bored, date and time concreted, elevation of piling platform, depths (from piling platform level) to pile base and to concrete cut-off level, length of casing used, detailed of reinforcement, brief description of soils encountered in the bore and details of any unusual occurrences during construction.

If a large number of piles are to be installed, it may be possible to optimize the design on the basis of pile load test.

B.3 CONSTRUCTION EXCAVATIONS

Construction should be in accordance with good practice and comply with the requirements of the responsible agencies.

All excavations greater than 1.5 m deep should be sloped or shored for worker protection.

Shallow excavations up to 3 m depth may use temporary side slopes of 1H:1V. A flatter slope of 2H:1V should be used if groundwater is encountered. Localized sloughing can be expected from these slopes.

Deep excavations or trenches may require temporary support if space limitations or economic considerations preclude the use of sloped excavations.

For excavations greater than 3 m depth, temporary support should be designed by a qualified geotechnical engineer. The design and proposed installation and construction procedures should be submitted to McIntosh•Lalani Engineering Ltd. for review.

The construction of a temporary support system should be monitored. Detailed records should be taken of installation methods, materials, in situ conditions and the movement of the system. If anchors are used, they should be load tested. McIntosh•Lalani Engineering Ltd. can provide further information on monitoring and testing procedures, if required.

Attention should be paid to structures or buried service lines close to the excavation. For structures, a general guideline is that if a line projected down at 45° from a horizontal, from the base of foundations of adjacent structures, intersects the extent of the proposed excavation, then these structures may require underpinning or special shoring techniques to avoid damaging earth movements. The need for any underpinning or special shoring techniques and the scope of monitoring required can be determined when details of the service ducts and vaults, foundation configuration of existing buildings and final design excavation levels are known.

No surface surcharges should be placed closer to the edge of the excavation than a distance equal to the depth of the excavation, unless the excavation support system has been designed to accommodate such surcharge.

B.4 FLOOR SLABS-ON-GRADE

All soft, loose or organic material should be removed from beneath slab areas. If any local hard spots such as old basement walls are revealed beneath the slab area, these should be over-excavated and removed to not less than 0.9 m below underside of slab level. The exposed soil should be proof-rolled and the final grade restored by general engineered fill placement. If proof-rolling reveals any soft or loose spots, these should be excavated and the desired grade restored by general engineered fill placement. Proof-rolling should be carried out in accordance with the recommendations given elsewhere in this Appendix. The subgrade should be compacted to a depth of not less than 0.3 m to density of not less than 95 percent Standard Proctor Maximum Dry Density (ASTM Test Method D698).

If for economic reasons, it is considered desirable to leave low quality material in place beneath a slab-on-grade, special ground treatment procedures may be considered. McIntosh•Lalani Engineering Ltd. could provide additional advice on this aspect, if required.

A leveling course of at least 150 mm in compacted thickness is recommended directly beneath all slabs-on-grade. For slabs in buildings requiring a subfloor depressurization system for soil gas control, the underslab gravels should consist of an open graded clean gravel with limited fine grained inclusions to allow free flow of gasses. The Class IA material (drainage gravel) is a suitable material for this application. Where these gravels are placed on top of fine grained soils, a geotextile filter fabric should be placed between the gravel and subgrade soils. Geotextile filter fabric is also recommended between the gravels and the polymer vapour barrier to protect the polymer from punctures. Where no subfloor depressurization system is required, the levelling course may consist of structural fill. Alternatively, a minimum thickness of 150 mm of pit-run gravel overlain by a minimum thickness of 50 mm of crushed gravel may be used. Very coarse material (larger than 25 mm diameter) should be avoided directly beneath the slabs-on-grade to limit potential stress concentrations within the slab.

General engineered fill, structural fill, pit-run gravel and crushed gravel are defined under the heading “Backfill Materials and Compaction” elsewhere in this Appendix.

The slab should be structurally independent from walls and columns supported on foundations. This is to reduce any structural distress that may occur as a result of differential soil movements. If it is intended to place any internal non-load bearing partition walls directly on a slab-on-grade, such walls should be structurally independent from other elements of the building founded on a conventional foundation system so that some relative vertical movement of the walls can occur freely.

The excavated subgrade beneath slabs-on-grade should be protected at all times from rain, snow, freezing temperatures, excessive drying and the ingress of free water. This applies during and after the construction period.

A minimum slab concrete thickness of 100 mm is recommended. Control joints should be provided in all slabs. Typically for a 125 mm slab thickness, control joints should be placed on a 3 m square grid, should be sawn to a depth of one-quarter the slab thickness and have a width of approximately 3 mm.

Wire mesh reinforcement, 150 mm square grid, should be provided to reduce the possibility of uncontrolled slab cracking. The mesh should be adequately supported and should be located at or above mid-height of the slab with adequate cover.

B.5 PROOF-ROLLING

Proof-rolling is method of detecting soft areas in an “as-excavated” subgrade for fill, pavement, floor or foundations or detecting non-uniformity of compacted embankment. The intent is to detect soft areas or areas of low shear strength not otherwise revealed by means of test holes, density testing or visual examination of the site surface and to check that any fill placed or subgrade meets the necessary design strength requirements.

Proof-rolling should be observed by qualified geotechnical personnel.

Proof-rolling is generally accomplished by the use of a heavy (15-60 tonne) rubber-tired roller having found wheels abreast on independent axles with high contact wheel pressures [inflation pressures ranging from 550 kPa (80 psi) up to 1,030 kPa (150 psi)].

A heavily-loaded truck may be used in lieu of the equipment described in the paragraph above. The truck should be loaded to approximately 10 tonnes (22,000 lbs) per axle and a minimum tire pressure of 550 kPa (80 psi).

Ground speed to be maximum of 8 km/hr (133 m/min) (5 mph) (400 ft/min). Recommended speed is 4 km/hr (65 m/min) (2.5 mph) (200 ft/min).

The recommended procedures is two complete coverages with the Proof-rolling equipment in one direction and a second series of two coverages made at right angles to the first series; one “coverage” means that every point of the proof-rolled surface has been subjected to the tire pressure of a loaded wheel. Less rigorous procedures may be acceptable under certain conditions subject to the approval of an engineer.

Any soft areas rutted or displaced materials detected should be either recompacted with additional fill or the existing material removed and replaced with general engineered fill or properly moisture conditioned as necessary.

The surface of the grade under the action of the proof-rolled should be observed, noting visible deflection and rebound of the surface or shear failure in the surface of granular soils as ridging between wheel tracks.

If any part of an area indicates significantly more distress than other parts, the cause should be investigated, by, for example, shallow auger holes.

In the case of granular subgrades, distress will generally consist of either compression due to insufficient compaction or shearing under the tires. In the first case, proof-rolling should be continued until no further compression occurs. In the second case, the tire pressure should be reduced to a point where the subgrade can carry the load without significant deflection and subsequently, gradually increased to its specified pressure as the subgrade increases in shear strength under this compaction.

B.6 SHALLOW FOUNDATIONS

Design and construction of shallow foundations should comply with relevant Building Code requirements.

The term “shallow foundations” includes strip and spread footings, mat slab and raft foundations.

Minimum footing dimensions in plan should be 0.45 m for strip footings and 0.9 m for square footings.

No loose, disturbed or sloughed material should be allowed to remain in open foundation excavations. Hand cleaning should be undertaken to prepare an acceptable bearing surface. Recompaction of disturbed or loosened bearing surface may be required.

Foundation excavation and bearing surfaces should be protected from rain, snow, freezing temperatures, drying and the ingress of free water, during and after footing construction.

Footing excavations should be carried down into the designated bearing stratum.

After the bearing surface is approved, a mud slab should be poured to protect the soil and provide a working surface for construction, should immediate foundation construction not be intended.

All constructed foundations should be placed on unfrozen soils, which should be at all times protected from frost penetration.

All foundation excavations and bearing surface should be observed by a qualified geotechnical engineer to confirm that the recommendations contained in this report have been followed and that soil conditions are consistent with those assumed in the design.

Where over-excavation has been carried out through a weak or unsuitable stratum in order to reach a suitable bearing stratum; or where a foundation pad is to be placed above stripped natural ground surface, lean-mix concrete or structural fill may be used to reinstate the grade. These materials are defined under the separate heading “Backfill Materials and Compaction.”

B.7 DRIVEN STEEL PILES

Full time observation of pile driving should be carried out by qualified geotechnical personnel.

Piles should initially be designed for minimum section and embedment on the basis of static design loads and shaft resistance.

Final design of driven steel piles could be carried out using a wave equation analysis. Design by this method would enable an optimum match of hammer type and weight to pile type and soil conditions and allow a check to be made on driving stresses.

Nominal Pile Size	Approximate Driving Energy	Final Set Blows Per 25 mm (1") For Last 25 mm (1")
250 mm (10")	37,000 J (27,500 ft.lbs)	20
	55,000 J (40,000 ft.lbs)	15
360 mm (14")	55,000 J (40,000 ft.lbs)	20

Steel piles should conform to the requirements of the applicable Building Code. When steel pipe piles are filled with concrete, it should conform to the requirements of the applicable Building Code, but should be of sufficient slump (150 mm or greater) to prevent voids forming and its consistency should be such as to prevent segregation.

Driving records should be kept for each pile. Information to be recorded should include pile dimensions, hammer type, rated energy, ram weight, cap block weight and type, anvil weight, number of blows for each 0.3 m of penetration and final set.

The elevation of the tops of driven piles should be measured immediately after driving. If uplift occurs in any piles during the driving of adjacent piles, the displaced piles should be re-driven to at least their previous final elevation and final set.

Piles should be spaced no closer than 2.5 times the pile diameter, measured centre-to-centre. Where piles are driven in groups, they should be driven from the centre outwards. In general, all piles in a group should be driven to approximately the same tip elevation.

If a group of four or more piles is required, group effects may reduce the working load of the pile group below calculated from the number of piles multiplied by the working load for an individual pile. If required, McIntosh•Lalani Engineering Ltd. can provide further design parameters for this case at the final design stage.

Strict control of pile location and orientation should be exercised to obtain accurate pile installation. Pre-boring of the surficial soils may be necessary to ensure proper location of the pile tip.

When piles are to be driven into very hard or frozen strata or boulders, special tips or pre-boring may be required.

For piles which will displace a significant amount of soil during driving, such as closed-end pipe piles, care should be taken that the driving will not cause strains of such magnitude as to cause damage to nearby structures.

Pile driving may result in significant vibrations which may be unacceptable for adjacent structures. In areas where this is a concern, continuous monitoring of vibrations induced in adjacent structures by a seismograph is recommended in order to assess the potential for damage and the need for modification of procedures.

If a large number of piles are to be installed, it may be possible to optimize the design on the basis of pile load tests.