

City of Winnipeg
Upgrades to the McPhillips and Tache
Pumping Stations
Tender Opportunity 1070-2019

APPENDIX E

McPhillips Geotech Report

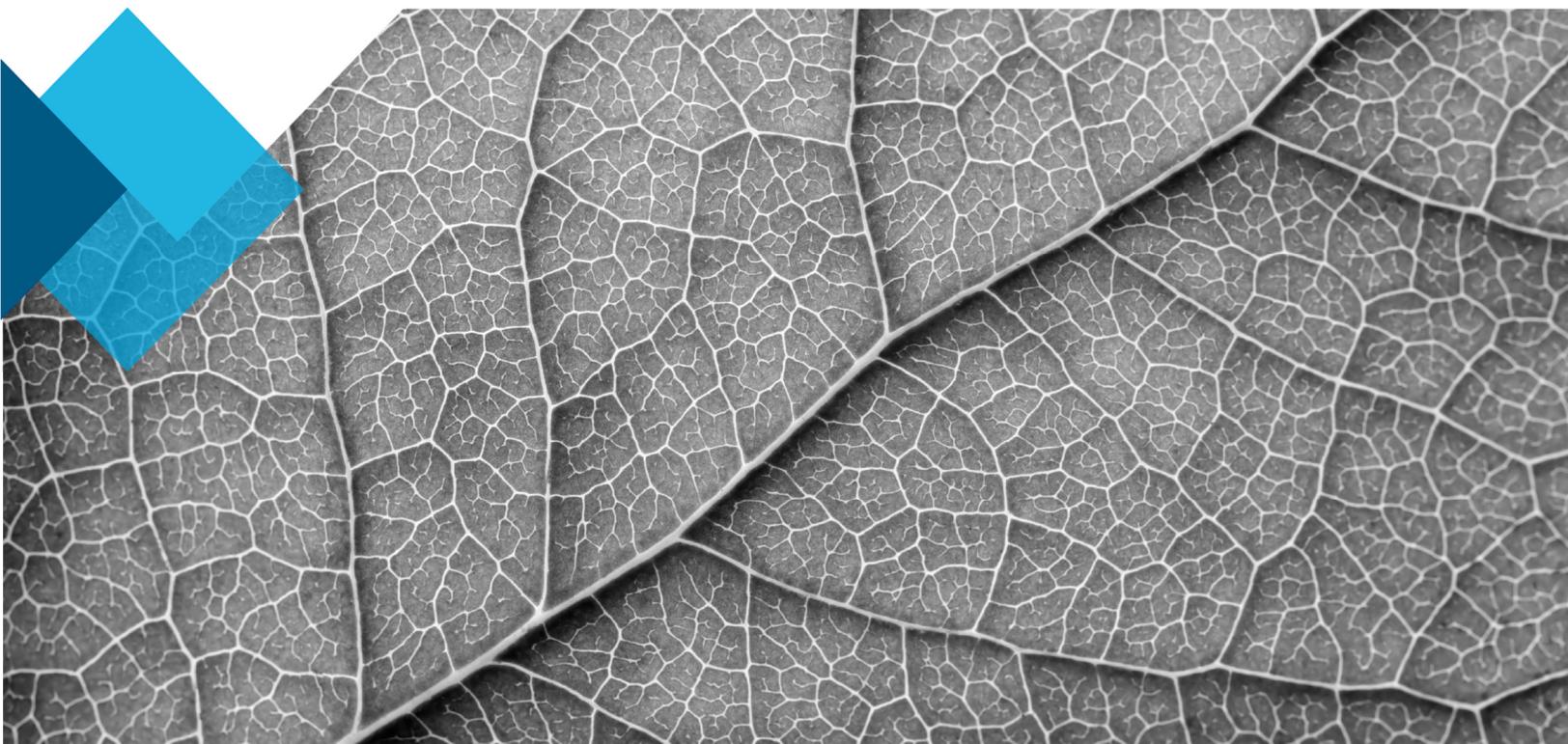


SNC • LAVALIN

Geotechnical Investigation McPhillips Pumping Station

Final Report

Prepared for: City of Winnipeg



Environment & Geoscience

November 6, 2019

Internal Ref: Project No. 661472-0000-4GER-0001_01

Notice to Reader

This report has been prepared and the work referred to in this report has been undertaken by SNC-Lavalin Inc. (SNC-Lavalin), for the exclusive use of the City of Winnipeg (the Client), under City of Winnipeg Project RFP 672-2018, who has been party to the development of the scope of work and understands its limitations. The methodology, findings, conclusions and recommendations in this report are based solely upon the scope of work for RFP 672-2018 and subject to the location, time and budgetary considerations described in the proposal and/or contract pursuant to which this report was issued. Any use, reliance on, or decision made by a third party based on this report is valid only for this specific project. SNC-Lavalin accepts no liability or responsibility for any damages that may be suffered or incurred by any third party as a result of the use of, reliance on, or any decision made based on this report for alternate projects.

The findings, conclusions and recommendations in this report (i) have been developed in a manner consistent with the level of skill normally exercised by professionals currently practicing under similar conditions in the area, and (ii) reflect SNC-Lavalin's best judgment based on information available at the time of preparation of this report. No other warranties, either expressed or implied, except those required by 672 2018 are made with respect to the professional services provided to the Client or the findings, conclusions and recommendations contained in this report. The findings and conclusions contained in this report are valid only as of the date of this report and may be based, in part, upon information provided by others. If any of the information is inaccurate, new information is discovered or project parameters change, modifications to this report may be necessary.

This report must be read as a whole, as sections taken out of context may be misleading. If discrepancies occur between the preliminary (draft) and final version of this report, it is the final version that takes precedence. Nothing in this report is intended to constitute or provide a legal opinion.

The contents of this report are confidential and proprietary. Other than by the Client, copying or distributing this report, in whole or in part, is not permitted without the express written permission of the Client or SNC-Lavalin.

SNC-Lavalin disclaims any liability to third parties in respect of the use of (publication, reference, quoting, or distribution), any decision made based on, or reliance on this report or any of its contents for any project other than that which it is intended for – City of Winnipeg RFP 672-2018.

Table of Contents

1	Introduction	1
2	Site Location and Description	1
3	Scope of Work	1
4	Geotechnical Investigation Details	1
4.1	Field Investigation	1
4.2	Geotechnical Laboratory Testing	2
5	Subsurface Conditions	2
5.1	Soil Profile	2
5.2	Groundwater, Seepage and Sloughing	2
5.3	Cobbles and Boulders	3
6	Geotechnical Recommendations	3
6.1	Geotechnical Considerations	3
6.2	Frost	3
6.2.1	Frost Penetration Depth	3
6.2.2	Procedures to Mitigate Frost Action in Buried Utilities	4
6.2.3	Frost Action and Foundations	4
6.3	Site Preparation	4
6.3.1	General	4
6.3.2	Proof Rolling	5
6.3.3	Subgrade Stabilization Techniques for Weak/Unstable Soils	5
6.3.3.1	Over-Excavation and Replacement	6
6.3.3.2	Geosynthetics	6
6.3.4	Temporary Excavations and Dewatering	7
6.4	Fill Materials, Placement and Compaction	8
6.4.1	General	8
6.4.2	Subgrade Fill	8
6.4.3	Structural Fill	8
6.4.4	Utility Trench Backfill	9
6.4.5	Fill Settlement	9
6.5	Foundations	10
6.5.1	Limit States Design	10
6.5.1.1	General	10
6.5.1.2	ULS Geotechnical Resistance Factors	11
6.5.2	Design Depths	11
6.6	Grade Supported Concrete Slabs and Foundations	11
6.6.1	Unheated Slabs/Foundations	13
6.6.2	Modulus of Subgrade Reaction, K_s	13
6.6.3	Foundation Drainage	13

6.7	Buoyancy Forces	14
6.8	Foundation Concrete.....	14
7	Construction Control and Monitoring	15
8	Disclosure of Information and Closure	16

Tables

Table 6-1	Estimated frost penetration depth under various surface covers.....	3
Table 6-2	Aggregate gradation specifications.....	9
Table 6-3	Estimated fill settlement versus compaction level.....	10

Appendices

I	Site Plan – Borehole Locations
II	Borehole Logs
III	Laboratory Test Reports

1 Introduction

This report presents the results of the geotechnical investigation conducted by SNC-Lavalin for the proposed slab-on-grade concrete foundation to be constructed for the McPhillips Pumping Station located in Winnipeg, Manitoba. The geotechnical investigation included the drilling program, field and laboratory soil testing, and preparation of a report that provides geotechnical recommendations for design and construction of the proposed development.

2 Site Location and Description

The proposed slab-on-grade concrete foundation is located at 360 McPhillips Street, Winnipeg, Manitoba. The general topography of the site was flat. The site was bounded by a building to the north, asphalt roadways to the west and south, and open grass covered terrain the east.

The site layout is shown in the site plan, [Figure I.1, Appendix I](#).

3 Scope of Work

The objective of the geotechnical investigation was to conduct a field investigation at the proposed development site and to provide geotechnical recommendations to support the design of the proposed McPhillips Pumping Station construction project. The following scope of work was completed:

- › A drilling program consisting of one (1) borehole drilled to a depth of 14.6 meters below ground surface (mbgs);
- › A field investigation consisting of geotechnical field tests, logging of soils and collection of soil samples for laboratory testing;
- › Laboratory testing of select soil samples obtained from the boreholes, including water contents, bulk unit weights, Atterberg limits, and grain size distribution analysis; and,
- › Preparation of a report summarizing the field investigation and providing geotechnical recommendations for development and design of the proposed development.

4 Geotechnical Investigation Details

4.1 Field Investigation

The field investigation was conducted on 27 September 2019. Maple Leaf Drilling from Winnipeg, Manitoba utilized a truck-mounted Mobile B40 drill rig equipped with continuous flight, solid stem augers with an automatic Standard Penetration Test (SPT) hammer to drill the boreholes.

The borehole location is presented on the site plan, [Figure I.1, Appendix I](#).

Disturbed soil samples were collected from the auger cuttings (grab samples) and from the SPT sampler. All soil samples were transported to the SNC-Lavalin soil testing laboratory in Saskatoon, Saskatchewan. The soil samples were stored in a humidity-controlled room to prevent drying prior to testing. Soil sample details (sample name, depth and type) are shown on the borehole logs in [Appendix II](#).

Field testing included Pocket Penetrometer tests conducted on all cohesive samples collected and SPT's conducted at selected depths. The results of field tests are presented on respective borehole records in [Appendix II](#). The Terms and Symbols used on the borehole logs are provided in [Appendix II](#) preceding the borehole logs.

The borehole was backfilled with auger cuttings and a bentonite chip cap at surface.

4.2 Geotechnical Laboratory Testing

Geotechnical laboratory tests were conducted on soil samples obtained from the borehole. The laboratory analyses included water contents, bulk unit weights, Atterberg limits, and grain size distribution analysis. The detailed laboratory test results are provided in [Appendix III](#). Select laboratory test results are also annotated on the borehole log presented in [Appendix II](#).

5 Subsurface Conditions

The soil and rock descriptions provided in this report are based on accepted standard methods of classification and identification routinely used in current geotechnical state of practice. A detailed description of the subsurface conditions encountered in the borehole is presented on the borehole log shown in [Appendix II](#). The stratigraphic boundaries shown on the borehole log are inferred from continuous sampling, observations during drilling, and the results of the SPT's. These boundaries represent transitions between soil types rather than exact planes of geological change.

5.1 Soil Profile

The general soil profile consisted of clay till from surface to a depth of 3.8 mbgs, followed by silty clay to a termination depth of 14.6 mbgs.

The till deposit was stiff to very stiff, medium plasticity, and moist. The silty clay deposit was firm to stiff, high plasticity, and moist to wet.

5.2 Groundwater, Seepage and Sloughing

Groundwater seepage conditions were encountered during test drilling. The depth of seepage conditions is shown on the borehole log provided in [Appendix II](#). Higher and potentially perched groundwater levels should be expected during or following spring thaw or periods of precipitation, and groundwater levels will fluctuate seasonally.

Sloughing conditions were not encountered and the borehole remained open immediately after drilling.

5.3 Cobbles and Boulders

Cobbles and boulders were encountered throughout the depth of exploration. Till is comprised of a heterogeneous mixture of clay, silt, sand and gravel-sized particles. Due to the nature of formation, transportation and deposition, till also inherently contains larger particle sizes (cobbles and boulders). Cobbles and boulders are often located randomly within till deposits but can also form concentrated layers, such as boulder pavements. The actual location and frequency of cobbles and boulders varies and the probability of encountering such deposits increases with the number of holes drilled, volume of soil excavated, number of piles installed, etc. Considering this, cobbles and boulders should be anticipated during construction. It is not possible to estimate the frequency/intensity of cobbles/boulders that may be encountered, but boulder sizes could range from 300 mm to as large as several metres in size (in rare cases).

6 Geotechnical Recommendations

6.1 Geotechnical Considerations

It is understood that a grade supported concrete slab will be constructed at the site. Over excavation and replacement may be required to achieve uniform soil conditions below the slab. If differential movements/floor cracking cannot be tolerated, then a structural slab should be constructed.

Design recommendations have been presented for frost action; site preparation; fill materials, placement and compaction; foundations; grade supported concrete slabs and foundations; buoyancy forces; and, foundation concrete.

6.2 Frost

6.2.1 Frost Penetration Depth

The till subgrade soils are considered to be susceptible to frost heaving if provided access to water. According to U.S. Corps of Engineers (USACE) Frost Design Soil Classification, the till soils can be classified as F3 or F4.

The estimated frost depth of the subgrade soils was calculated using the modified Berggren equation provided in the Canadian Foundation Engineering Manual (CFEM) under various surface covers. The estimated frost penetration depths are summarized in Table 6-1

Table 6-1 Estimated frost penetration depth under various surface covers.

Surface Cover	Design Return Period		
	Normal	10 Year Extreme	50 Year Extreme
Estimated Frost Penetration Depth (m)			
Gravel/Asphalt	2.5	2.6	3.0
Vegetated	1.9	2.0	2.3
Concrete	2.4	2.5	2.8

Frost depths are applicable to unheated areas. It is noted that frost depths will increase where granular fills are used.

6.2.2 Procedures to Mitigate Frost Action in Buried Utilities

The native soil near ground surface is considered to be frost susceptible. SNC-Lavalin recommends that buried utilities that are frost sensitive should have a minimum soil cover of at least 0.5 m greater than the anticipated frost penetration depth. Frost sensitive utilities buried with less than the recommended soil cover should be protected with rigid polystyrene insulation to avoid frost effects that may cause damage to the utility pipes. Rigid insulation placed under areas subject to vehicular wheel loads should be provided with a minimum cover of 600 mm of compacted granular base and/or pavement. The design of the insulation system (depth, extent, thickness, etc.) will depend on several factors and should be determined in consultation with a qualified geotechnical consultant.

6.2.3 Frost Action and Foundations

The volume increase that occurs when water changes to ice is one of the causes of frost heave. However, it is also recognized that a phenomenon known as ice segregation is the predominant mechanism: Water is drawn from unfrozen soil to the freezing zone where it accumulates to form layers of ice, forcing soil particles apart and causing the soil surface to heave. The magnitude of frost heave due to ice segregation can be much more severe than that of a simple state change in the soil pore water. As such, movement sensitive foundations should be founded below the depth of frost penetration. Alternatively, measures to prevent ice segregation must be taken (i.e., dewatering, insulation, heating the area, replacement of frost prone soil with stable fill, etc.). Such measures (if required) should be designed in consultation with a suitably qualified geotechnical consultant. The subsequent thawing process can result in severe thaw weakening of any subgrade when the excess water is trapped between the surface loading, such as a vehicle, and the still frozen material deeper down in the subgrade. Significant subgrade deterioration can also result from freeze/thaw cycles.

A different form of frost action, called 'adfreezing', occurs when soil freezes to the surface of a foundation. Heaving pressures developing at the base of the freezing zone are transmitted through the adfreezing bond to the foundation, producing uplift forces capable of appreciable vertical displacements. Relatively little is known of the magnitude of the forces that may be generated, but bond strengths of adfreezing in the order of 100 kPa for steel surfaces and 70 kPa for wood and concrete have been measured. Providing a bond breaker between the foundation and the soil can reduce the potential for foundation movements due to adfreezing forces.

6.3 Site Preparation

6.3.1 General

A proper site drainage management plan should be prepared and implemented prior to commencing any site work. The existing, local drainage pattern should be preserved, if reasonably possible, as changing the local drainage pattern can have disruptive and adverse consequences. Excess water should be drained from the work areas as quickly as possible both during and after construction. Initial grading operations should be focused on providing surface drainage, such that precipitation and surface run-off is directed away from work areas.

Following stripping of topsoil and excavation to design subgrade elevation, the exposed subgrade should be inspected by qualified geotechnical personnel to verify the removal of unsuitable materials and to provide additional recommendations, as appropriate. Unsuitable materials include topsoil, organic matter, frozen material, vegetation, oversized material and other deleterious materials. The lateral extent of all excavations and removals should be at least 1.5 m from beyond the edge of all structures. Topsoil may be stockpiled in an approved location and re-used for non-structural areas only, such as landscaping.

As a minimum (unless otherwise stated), all exposed subgrade soil within the proposed development areas should be scarified to a minimum depth of 150 mm, moisture conditioned (wetted or dried) to within $\pm 2\%$ of optimum moisture content and compacted to at least 98% of Standard Proctor Maximum Dry Density (SPMDD) tested in accordance with ASTM Method D 698. Subgrade preparation should not be performed on very soft, loose or wet subgrade as construction equipment may further weaken the subgrade. If weak soil conditions are encountered and scarification/compaction is not practical, subgrade stabilization techniques will be required (as discussed in the following section).

6.3.2 Proof Rolling

Upon completion of initial site preparation activities (as discussed above), proof rolling of the subgrade should be conducted to verify that competent and uniform soil subgrade support conditions have been achieved. Proof rolling should not be conducted during or shortly following precipitation events, and heavy equipment shall not be allowed to travel on wet/soft subgrade soils until adequate drying has occurred. Proof rolling should be performed by two passes of a dual-wheel truck (or comparable equipment) with a minimum of 80 kN single axle load. Soils which display rutting or appreciable deflections upon proof-rolling may require additional treatment, as discussed below. Following efforts to stabilize the soil, proof rolling should be repeated. All proof rolling and compaction efforts should include documentation detailing the findings, including photographs where possible. All finished subgrades should be protected from construction traffic and erosion as soon as possible.

6.3.3 Subgrade Stabilization Techniques for Weak/Unstable Soils

Where suitable soil conditions exist, “typical” site preparation (i.e., scarification, moisture conditioning and re-compaction), as discussed above, is generally suitable to provide a stable working base for construction equipment and is usually the most economical site preparation alternative. However, where unsuitable and/or soft subgrade conditions exist, “typical” site preparation can be very difficult (impractical in many cases), particularly where high groundwater conditions exist. Adverse weather conditions, such as excessively wet and dry (drought) conditions, can also significantly impact the effectiveness of “typical” site preparation. Where time constraints exist, “typical” site preparation methods may not allow for completion of construction within the available time frame. Where a combination of these limitations exists, alternate site preparation techniques are generally utilized.

Alternate site preparation techniques generally consist of additional over-excavation and replacement of soft/unstable subgrade soils with suitable fill material, and/or utilization of geosynthetics to bridge/stabilize soft/unstable soils.

The following general recommendations should be utilized as a guide where unsuitable and/or soft soil conditions are encountered and where “typical” site preparation methods are not suitable. On-site supervision and direction will be required to provide site-specific recommendations depending on actual site conditions.

6.3.3.1 Over-Excavation and Replacement

Completely remove all unsuitable materials from the construction areas (topsoil, vegetation, organic matter, debris and other deleterious materials). Where the depth of soft/unstable soils is not excessive, it may be feasible to completely over-excavate (to competent subgrade soil) and replace the soft/unstable soils. Upon reaching competent subgrade soil, “typical” site preparation methods (as described above) may be utilized. Suitable, approved fill material may be utilized to replace the over-excavated soil. Where the depth of soft/unsuitable soils is excessive, complete over-excavation and replacement may not be feasible or economical. In these areas, the depth of over-excavation should be limited and geosynthetics should be utilized to facilitate construction, as discussed below. Over-excavated areas backfilled with granular fill should be provided with a sub drain system.

6.3.3.2 Geosynthetics

Geosynthetics (woven geotextile, geogrid in conjunction with non-woven geotextile or combination geotextile/geogrid products) are commonly used where soft/unstable soil conditions are encountered to provide material separation and stabilization of the subgrade soils. Geosynthetics are varied in purpose/cost, but in general the effectiveness/cost increases from non-woven geotextile (primarily for material separation purposes) to woven geotextile (provides material separation as well as some strength gain) to geogrid products (primarily for enhanced strength gain and reduction in required aggregate thickness). Geosynthetics are also often utilized over suitable subgrade soils to provide material separation and to enhance the long-term performance (“value-added” construction).

Unsuitable materials should be completely removed from the construction areas (topsoil, vegetation, frozen soil, organic matter, debris and other deleterious materials). The subgrade soils should be over-excavated to a minimum depth of 450 to 600 mm below the design subgrade elevation. The actual depth of over-excavation and type of geosynthetic product should be determined based on-site conditions and site-specific requirements. It is strongly recommended that a small “test section” should be constructed to confirm that stable subgrade conditions are attainable prior to over-excavation of a large area.

Where soft subgrades with a CBR of about 3% or less exist, or where the subgrade soils are susceptible to pumping into the granular fill under traffic loading, nonwoven geosynthetics should be used for separation/filtration in combination with geogrids to increase bearing capacity. Alternately, approved combination geotextile/geogrid products may be utilized (i.e., Combigrid or similar). Woven geotextiles or geogrids (without accompanying non-woven geosynthetics) may be used for reinforcement where separation/filtration is not of concern and the subgrade CBR is greater than 3%. The geosynthetics should be placed by hand or with low contact pressure equipment in accordance with the manufacturer’s recommendations utilizing appropriate overlaps at joints.

Where geosynthetics are utilized over soft/unstable subgrade soils, subgrade preparation should be limited to leveling the subgrade surface (to minimize disturbance of the soils). Soft/unstable soils typically have very poor trafficability; as such, long reach and/or low contact pressure construction equipment should be utilized.

Where geosynthetics are utilized, granular fill materials are recommended. The first lift of granular fill placed on the geosynthetic should be at least 150 to 300 mm in thickness (depending on the severity of the soft/unstable subgrade soils) and should be placed by end-doze and spread methods. Construction equipment shall not be allowed to travel over the geosynthetic until the first lift of granular fill has been placed. The first lift of granular fill should be as clean as possible (preferably less than 5% fines) to minimize the potential for softening due to the upward migration of moisture from the underlying soft/unstable soils.

Wherever possible, this initial layer of granular fill should be graded to daylight and thus allow for drainage. Where this is not possible, subsurface drainage systems should be installed to collect and discharge water (if feasible). If drainage is not feasible, as a minimum, the subgrade surface should be graded to direct water that accumulates within the granular fill to the outer edges of the work area.

Depending on the position of the groundwater table, the site-specific soil conditions and the fill materials utilized, it may be necessary to compact the initial lifts of fill with static compaction equipment to minimize disturbance of the underlying soils and capillary rise of groundwater into the fill materials. Upon adequate “bridging” of the soft/unstable soils, vibratory compaction may be utilized. As mentioned above, it is recommended that a small “test section” should be constructed to confirm that the depth of over-excavation, thickness of the initial lifts of fill, fill material type and chosen geosynthetic product are appropriate to create a stable working surface. If a stable working surface does not exist upon placement and compaction of the initial lifts of fill, it may be necessary to increase the depth of over-excavation, increase the thickness of the initial lifts of fill, use alternate fill materials, utilize a stronger geosynthetic (i.e., geogrid vs. geotextile) or provide a second layer of geosynthetic. Once the appropriate construction technique has been established, the remainder of the areas may be constructed.

6.3.4 Temporary Excavations and Dewatering

The temporary slope angle of the excavations shall follow the recommendation stated in the Occupation Health and Safety (OH&S) Regulations, 1996. Soil types were classified according to subsections (3) and (4) of part 260 of the OH&S Regulations. Within the anticipated depth of excavation, the firm to stiff till soils would be classified as Type 3 soils. The maximum slope angle for temporary excavations in Type 3 soils shall be 1H:1V (45°). Shallower side slopes may be required below the groundwater table or where soft soil conditions are encountered. Variability in surface soils exists, and it is recommended that qualified geotechnical personnel conduct an inspection of any excavations prior to workers entering the excavated area, and written records of the inspections be maintained. The excavation slopes should be checked regularly for signs of spalling, cracking, tension cracks at crest, etc., particularly after periods of rain. Local flattening of the excavation slopes may be required if instabilities of the cut slopes are observed.

For temporary excavations equipment, spoil piles, rocks, and construction materials should be kept at least 1 m from the edge of the excavation as stated in OH&S Regulation’s part 260(1). For excavations that will remain open for a relatively long duration of time, it is recommended that the stockpiling distance from the crest of the excavation should be equal to or greater than the depth of excavation.

Drainage trenches with periodic low points for standard sump pumps should be sufficient for dewatering shallow excavations at this site. As it is difficult to estimate the amount of water that will be encountered, close monitoring of groundwater ingress into the excavations is recommended. Other dewatering methods may be required if conventional methods prove to be insufficient. Surface drainage should be directed away from the crest of any excavation, particularly where workers and equipment will be present.

Excavations that are made close to and beneath the level or elevation of existing footings, structures or utilities should be avoided if possible (if applicable). Where such excavations are unavoidable, the temporary excavation should be cut as outlined above, extending from a point at least 0.5 m away from the base of the existing footing/structure/utility. No vertical unsupported cuts shall be made. Shoring systems may be required in some areas. The design of the shoring system and all excavations adjacent to existing footings, structures or utilities should be reviewed and monitored by qualified geotechnical personnel.

6.4 Fill Materials, Placement and Compaction

6.4.1 General

All proposed fill material should comply with the recommendations provided in this report and should be approved by qualified geotechnical personnel prior to use. All fill soils should be free of appreciable amounts of deleterious and/or organic materials, large particle sizes and contaminants. Fill soils should not be placed in a frozen state or placed on a frozen subgrade. All lumps of materials should be broken down during placement.

Prior to placement of fill material, representative bulk samples (about 25 kg) should be taken of the proposed fill soils and laboratory tests should be conducted to determine (as applicable) Atterberg limits, natural moisture content, grain size distribution and standard Proctor moisture density relationship. These test results will be necessary for the proper control of construction for the engineered fill.

Prior to placing any fill, the exposed subgrade surface should be prepared in accordance with the preceding sections. It is important that the fill soils be compacted uniformly in order to maintain uniformity and minimize the potential of subsequent differential vertical movements.

6.4.2 Subgrade Fill

Subgrade fill, if required to achieve a uniformly level subgrade surface, should be placed in loose lifts (150 mm thickness, maximum). Subgrade fill, if required, should consist of soil free of unsuitable materials (topsoil, organic matter, vegetation, frozen soil, oversized material and other deleterious materials). Locally available glacial deposits should be moisture conditioned (wetted or dried) to within $\pm 2\%$ of optimum moisture content and compacted to at least 98% of SPMDD tested in accordance with ASTM Method D 698.

6.4.3 Structural Fill

Well-graded, clean granular material (gravel or crushed rock) is preferred as structural fill at this site due to the relative ease of compaction and more uniform/rapid settlement response (as compared to poorly graded granular soils or fine-grained soils). If the use of well graded, clean granular fill is cost prohibitive, then the use of consistent, locally available till deposits may be permissible; clay soils are not recommended for use as structural fill. The settlement response of non-granular materials or poorly graded granular materials will be less uniform and will take longer to develop as compared to well graded granular materials. Additional time and effort will also be required to moisture condition and place these materials. Beneath hard-surfaced, grade-supported structures, a nominal thickness of structural granular fill (base course and sub-base course materials) will be required.

All structural fill should be placed in thin lifts (150 mm thickness, maximum), moisture conditioned (wetted or dried) to within $\pm 2\%$ of optimum moisture content, and uniformly compacted to at least 100% of SPMDD tested in accordance with ASTM Method D 698. Where not contained by grade beams or suitable curbs, the structural fill should extend laterally 1 m or equal to the full depth of fill (whichever is the greater) beyond the footprint of grade-supported structures (asphalt surfacing, concrete slabs etc.).

The recommended aggregate gradation requirements have been presented in [Table 6-2](#). Alternate gradations may be acceptable but should be approved by qualified geotechnical personnel prior to use.

For granular sub-base course material, the uppermost 300 mm of the fill should meet the gradation requirements presented above. For lower levels of sub-base fill, over-sized particles may be incorporated. For quality control testing of fill material containing over-sized particles, the gradation should be determined on samples with all oversized materials (i.e., greater than 50 mm) removed.

Table 6-2 Aggregate gradation specifications.

Sieve Size	Percent Passing by Weight				
	Base Course Type 33	Base Course Type 31	Sub-Base Type 6	Sub-Base Type 8	Traffic Gravel
50 mm			100	100	
31.5 mm		100			
18 mm	100	75 - 90			100
12.5 mm	75 - 100	65 - 83			
5 mm	50 - 75	40 - 69			45 - 80
2 mm	32 - 52	26 - 47	0 - 80	0 - 90	25 - 60
900 µm	20 - 35	17 - 32			
400 µm	15 - 25	12 - 22	0 - 45	0 - 60	0 - 30
160 µm	8 - 15	7 - 14	0 - 20	0 - 25	
71 µm	6 - 11	6 - 11	0 - 6	0 - 15	
Plasticity Index	0 - 6	0 - 7	0 - 6	0 - 6	
Fractured Face %	Min 50	1 Min 50			
Lightweight Pieces %	Max 5	Max 5			

¹ Adopted from Saskatchewan Highway and Transportation Design Manual.

² Type 31 base gravel should only be used when at least 25% of the aggregate particles (by weight) retained on the 5 mm sieve have one or more fractured faces.

6.4.4 Utility Trench Backfill

Utility bedding materials will vary depending on the type of utility. Utility bedding material gradation, placement, thickness, compaction, etc., should be in accordance with the utility manufacturer's specifications and recommendations. Care must be taken to ensure damage does not occur to the utilities as a result of placement/compaction of the bedding material and overlying fill materials.

Below buildings/structures and concrete surfaced areas, the use of well graded granular fill is recommended above the bedding material (as discussed above) as this type of material will settle less and more uniformly as compared to common fill (i.e., locally excavated soil). Within all other areas (where some potential settlement of the excavation backfill material may be permissible), the use of locally excavated soil as backfill should be suitable.

6.4.5 Fill Settlement

Fill materials will tend to settle due to self-weight and any imposed loading. The amount of settlement is unpredictable due to a number of variables associated with the properties of the fill material and the placement history of the fill. The settlement of fill materials can be reduced by adhering to strict placement and compaction specifications for the entire fill thickness (i.e., utilizing thin uniform lifts, maintaining moisture content near optimum, compacting to a uniform, high density condition). Maintaining a uniform fill thickness will also serve to minimize differential movements across the fill area. The estimated settlements of cohesive and non-cohesive fill materials as a function of compaction level have been presented in Table 6-3.

Table 6-3 Estimated fill settlement versus compaction level.

Compaction Level (% SPMD)	Estimated Fill Settlement (% of Fill Thickness)	
	Cohesive Soils	Non-Cohesive Soils
100	0.5	<0.5
98-100	1.0	0.5
95-98	1.5	1.0
90-95	4.0	3.0
<90	>4.0	>3.0

¹ SPMD = Standard Proctor Maximum Dry Density ($\pm 2\%$ of optimum moisture content).

The above settlement estimates are for fill materials placed during non-freezing conditions. The self-weight induced settlement will be significantly higher than shown in Table 6-3 if frozen fill materials are utilized (particularly for cohesive fill materials).

6.5 Foundations

6.5.1 Limit States Design

6.5.1.1 General

As per limits states design principles presented in the Canadian Foundation Engineering Manual (4th edition, 2006), foundation design must consider both ultimate limit states (ULS) and serviceability limit states (SLS).

ULS are primarily concerned with collapse mechanisms of the structure, and hence, safety. For foundation design, ULS consist of:

- › Exceeding the load carrying ability of the ground that supports the foundation (i.e., ultimate bearing capacity)
- › Sliding
- › Uplift
- › Overturning
- › Large deformation of the foundation subgrade that leads to an ULS being introduced in the structure
- › Loss of overall stability

SLS represent conditions or mechanisms that restrict or constrain the intended use, function or occupancy of the structure under expected service or working loads. SLS are usually associated with movements or deformations that interrupt or hinder the function (i.e., serviceability) of the structure. For foundation design, SLS generally consist of:

- › Excessive movements (e.g., settlement, differential settlement, heave, lateral movement, cracking, tilt)
- › Unacceptable vibrations
- › Local damage and deterioration

During the design process, the structural engineer will need to consider both ULS and SLS geotechnical parameters. Factored (ULS) structural loads will need to be compared to factored (ULS) geotechnical parameters. Likewise, working structural loads will need to be compared to SLS geotechnical parameters.

6.5.1.2 ULS Geotechnical Resistance Factors

For the purposes of this report, ultimate geotechnical design parameters have been presented. To determine factored parameters (limit states design), the ultimate parameters should be multiplied by the applicable geotechnical resistance factors (ϕ) as per the National Building Code of Canada 2015 (NBC). The recommended geotechnical resistance factors (ϕ) as per the National Building Code of Canada 2015 (NBC) are as follows:

1. Shallow Foundations

- a) Vertical bearing resistance from semi empirical analysis using laboratory and in situ test data ($\Phi = 0.5$)
- b) Sliding
 - a. Based on friction [$c = 0$] ($\Phi = 0.8$)
 - b. Based on cohesion/adhesion [$\tan \theta = 0$] ($\Phi = 0.6$)

6.5.2 Design Depths

For the purposes of this report, design depths have been referenced to existing ground surface; the structural designer must consider finished grade elevations relative to existing ground surface. If ground elevations are altered significantly from current levels, SNC-Lavalin should be consulted to confirm design parameters.

6.6 Grade Supported Concrete Slabs and Foundations

The near-surface subgrade soils consisted of firm to stiff glacial till. Grade-supported concrete slabs shall perform satisfactorily provided that some floor slab movements can be tolerated, and the recommendations presented in Sections 6.3 and 6.4 are adhered to. If some differential movements/floor cracking cannot be tolerated, then a structural slab should be constructed.

Providing positive drainage adjacent to buildings/slabs and extending downspouts well away from buildings/slabs will reduce the risk for soil moisture profile variations. The subgrade soils should not be allowed to become overly dry or overly wet during construction to minimize potential movements from wetting or drying in the future.

SNC-Lavalin recommends that measures should be taken to accommodate potential slab movements by constructing structural elements such as partition walls, staircases, grade beams, columns, etc. independent of the slab. We acknowledge that it is common practice by many structural engineers and contractors to rigidly connect grade-supported concrete slabs to structurally supported foundation elements via rebar in an effort to prevent formation of a vertical “lip” between the two structures. This practice creates stress concentrations as the supporting soils undergo volume changes (i.e., slab wants to move relative to stable structural element). If this is to be done, we recommend that a compressible void form (minimum of 100 mm of net void space) should be placed below the concrete slab for some distance away from the connection and that the structural connection should be adequate to withstand any vertical surcharge loads on the slab, or, that the structural connection should be adequate to withstand the imposed forces on the slab (which may be downward as a result of surcharge loading or upward as a result of vertical heave of the slab). It is noted that upward vertical pressures can be very significant (potentially in the order of 500 to 1,000 kPa), and that resisting these forces may not be feasible.

The following minimum recommendations should be incorporated into the design of reinforced, grade-supported, cast-in-place slabs at this site:

- › A serviceability limit states (SLS) bearing pressure of 160 kPa may be utilized for design of the slab-on-grade foundation (to limit foundation settlement to 25 mm or less).
- › A factored ultimate limit states (ULS) bearing pressure of 250 kPa may be used for design of the slab-on-grade foundation (i.e., bearing capacity against soil shear failure).
- › Over-excavate, as required, to shape the site to the design subgrade elevation (additional excavation required where fill materials are removed). Soft subgrade areas should be over-excavated and replaced with locally available soils (drier/stronger material from elsewhere on the site or reworked existing soils) or approved imported fill. Backfill materials should be moisture conditioned (wetted or dried) to within $\pm 2\%$ of optimum moisture content and uniformly compacted to at least 98% of standard Proctor density. Additional structural fill thickness and/or the use of geosynthetics may be required where soft soils are encountered to provide an adequate structure to 'bridge' underlying soft soils. Cover the prepared subgrade surface with approved structural fill as soon as practical to minimize the potential for moisture change within the subgrade soils.
- › Conduct site grading, as required, to allow for the placement of a uniform thickness of compacted structural fill between the underside of the grade supported slab and the prepared subgrade surface. A minimum structural fill thickness of 300 mm is recommended. The uppermost 150 mm of the fill should consist of crushed, granular base course material. Lower lifts of fill may consist of granular sub base material or approved fine grained soils.
- › All structural fill should be placed in thin lifts (150 mm thickness, maximum), moisture conditioned (wetted or dried) to within $\pm 2\%$ of optimum moisture content, and uniformly compacted to at least 100% of SPMDD tested in accordance with ASTM Method D 698.
- › Separation joints should be used to isolate the slab from foundation walls, columns, etc.
- › Reinforce the concrete slab and provide control joints at regular intervals to provide for controlled cracking.
- › The finished grade should be landscaped to provide for positive site drainage away from the structure.
- › Concrete slabs should not be constructed on loose, softened, desiccated, frozen or wet soil.
- › Frost should not be allowed to penetrate beneath the concrete slab just prior to, during or after construction.
- › Continuous quality control inspection by SNC-Lavalin should be provided during fill placement.

6.6.1 Unheated Slabs/Foundations

Grade supported concrete slabs exposed to freezing conditions will be subject to differential movements associated with frost action. The potential for differential movements associated with frost action can be minimized by over excavating and replacing a greater thickness of subgrade soil with low frost susceptible granular fill (in the order of 1,000 mm to 1,500 mm is recommended), by placing sub-horizontal rigid polystyrene insulation below the slabs and/or by providing minimal heat (five degrees Celsius minimum) and air circulation within the building during freezing conditions (if applicable). If insulation is to be utilized, the insulation should have a minimum thickness of 120 mm and should extend below the slab and sub-horizontally away from the outer edges of the slab a minimum distance of 2.4 m. The insulation should be covered with a minimum of 300 mm of soil cover to provide protection against damage and should be positively sloped away from the slab.

If differential movements cannot be tolerated, concrete slabs should be constructed as a structural slab (i.e., pile supported) over a compressible void form.

6.6.2 Modulus of Subgrade Reaction, K_s

The modulus of subgrade reaction is a conceptual relationship between soil pressure and deflection (ie, imposed pressure divided by the corresponding deflection), and varies with the intensity of the pressure, size of the loaded area and the thickness/properties of the supporting soils (sub slab granular fill as well as the underlying native soils). For the preliminary design of grade supported floor slabs at this site, a modulus of subgrade reaction in the order of 27 to 54 MN/m³ (100 to 200 pci) may be utilized. This preliminary modulus of subgrade reaction is considered applicable to small load contact areas, such as storage rack posts or wheel loads. For larger contact areas, the modulus of subgrade reaction will differ and should be determined in consultation with SNC-Lavalin. For a more accurate estimate of K_s , additional information will be required including the imposed load/pressure, size of loaded area and the thickness/type/density of sub slab granular fill. If a more accurate determination of K_s is required, then a plate bearing test is recommended.

6.6.3 Foundation Drainage

To minimize the potential for water collection below grade supported concrete slabs, the finished grade must be landscaped to provide for positive site drainage away from the structure. A perimeter weeping tile drainage system (installed at the base of the perimeter foundation) would reduce the potential for external water infiltration below the foundation/concrete slab. If a perimeter drainage system is utilized, the subgrade surface should be graded to provide drainage toward the perimeter drainage system. Where thick layers of granular fill are placed below the slab (potential receptor for water), proper exterior drainage is critical. In these cases (depending on the actual slab elevation and final exterior grades), it may be preferable to utilize a greater thickness of approved fine grained soils as structural fill and/or provide an internal drainage system to provide controlled collection and discharge of water that might accumulate below the slab. Many drainage system configurations are possible, but would generally consist of sloping the subgrade surface to collection points (i.e., sump pits) and/or utilizing perforated drainage pipe (weeping tile) to convey water to the collection points. The drainage system should be positively drained to sump pits equipped with automatic sump pumps (or gravity drained, if possible) which discharge water to a suitable location. Granular drainage materials should consist of well graded material with less than 3% fines (silt and clay sized particles) and should be separated from the subgrade soils with a continuous layer of high permeability, non-woven geotextile. Details for drainage systems should be reviewed by the geotechnical consultant prior to finalizing the design.



A sub-slab drainage system is recommended below all sub-surface slabs (if applicable) to allow for collection and controlled discharge of water which may enter or accumulate below the slab. The sub-slab drainage system should consist of a series of perforated drainage pipes, spaced at a maximum of 3 m to 4 m on centre, with the invert placed at the base of a continuous layer of clean, free-draining granular material (minimum of 350 mm below the underside of the slab). The thickness of the free-draining granular layer should be at least twice the height of the drainage pipe. The subgrade surface/perforated drainage pipes should be positively sloped to promote drainage toward sump pits. The granular drainage layer should be separated from the underlying subgrade soil with a continuous layer of high permeability, non-woven geotextile, capable of transmitting a flow of not less than 50 litres per second per square metre (ASTM D 4491). The clean, free-draining granular material should consist of well graded material with less than 3% fines (silt and clay sized particles). Alternately, the perforated drainage pipes could be installed in trenches. In this case, the drainage pipe invert elevation should be situated a minimum of 750 mm below the underside of the floor slab. The drainage pipe and clean drainage aggregate should be fully encapsulated in high permeability, non-woven geotextile, as described above. The sub-slab drainage system should be positively drained to sump pits equipped with automatic sump pumps and discharged in accordance with local regulations.

6.7 Buoyancy Forces

The groundwater level at the subject site was situated approximately 2.0 m below ground surface based on 27 September 2019 field results. If a drainage system is not installed below the proposed structures, it will be necessary to design the structure to resist uplift forces due to buoyancy. The uplift hydrostatic pressure acting on the base of the foundation element would be equal to the depth of the foundation element extending below the groundwater table multiplied by the unit weight of water (9.81 kN/m³).

6.8 Foundation Concrete

Water-soluble sulphates (gypsum crystals) are common in cohesive soil deposits in this area. Sulphate resistant cement is recommended for all foundation concrete in contact with the subgrade soils at this site. If imported fill material is utilized, it is recommended that the fill soil be tested for sulphate content to determine whether the above state recommendations remain valid. It should be noted that most fine-grained soils encountered in Manitoba contain concentrations of sulphates.

The recommendations stated above for the subsurface concrete at this time may require further additions and/or modifications due to structural, durability, service life or other considerations which are beyond the geotechnical scope. A designer competent in concrete mix design should complete the specifications for the concrete mix.

7 Construction Control and Monitoring

The recommendations presented in this report are based on the premise that full time inspection, monitoring, and control testing are provided by qualified SNC-Lavalin personnel during site development and construction, and that appropriate design considerations are given to local risk factors (slope stability, flow patterns, etc.) by qualified personnel. Hence, quality control should be provided as follows:

- › Inspection during site grading, clearing/excavation and proof rolling to verify the removal of unsuitable materials;
- › In-situ density and moisture content testing during subgrade preparation and placement of fill/backfill;
- › Inspection during foundation installations; and,
- › Materials and concrete laboratory testing during construction.

8 Disclosure of Information and Closure

The Client hereby agrees that any information provided in this report, even if it is identified as being supplied in confidence, may be disclosed where required by law or if required by order of a court. The proponent hereby consents to the disclosure, on a confidential basis, of this report by SNC-Lavalin Inc. to the Client's advisers retained for the purpose of evaluating or participating in the evaluation of this report.

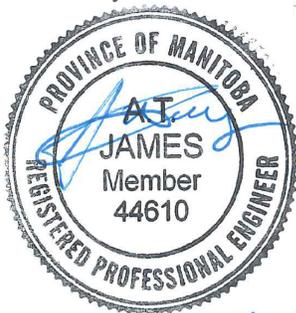
We trust that this report meets your requirements. Should you have any questions or comments please contact us at +1.306.668.6800.

Prepared by:



Nevin Cross
Geotechnical Engineer-In-Training

Reviewed by:



November 6th 2019

Alistair James, P. Eng.
Principle Geotechnical Engineer

Environment & Geoscience
Infrastructure

Appendix I

Site Plan – Borehole Locations



LOCATION PLAN
SCALE: 1:250,000



SITE PLAN
SCALE: 1:1,000

LEGEND

● BOREHOLE

NOTES

- COORDINATES ARE UTM NAD 83 ZONE 14.
- IMAGE FROM AUTODESK ONLINE MAPS (YEAR UNKNOWN).



DRAFT

REVISIONS

REV	DATE	DESCRIPTION	DES	DRN	CHK	APP
00	2019 10 02	-	NC	EO	-	-

CLIENT

CITY OF WINNIPEG

PROJECT LOCATION

MCPHILLIPS PUMPING STATION,
WINNIPEG, MB

REFERENCE DRAWINGS

DWG No	DESCRIPTION

TITLE

SITE PLAN
BOREHOLE LOCATION

DISCLAIMER

This drawing was prepared for the exclusive use of City of Winnipeg (the "Client"). Unless otherwise agreed in writing by SNC-Lavalin Inc., SNC-Lavalin Inc. does not accept and disclaims any and all liability or responsibility arising from any use of or reliance on this drawing by any third party or any modification or misuse of this drawing by the Client. This drawing is confidential and all intellectual property rights embodied or referenced in this drawing remain the property of such parties, as determined by the applicable services contract or contracts between SNC-Lavalin Inc. and the Client.

NOT FOR CONSTRUCTION

DATE	DWG No	FIG	REV
2019 10 02	661472-0000-4GDD-0101		

Appendix II

Terms and Symbols and Borehole Logs



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

- 1) Lithology/ Texture
- 2) Colour and Oxidation
- 3) Consistency and Condition
- 4) Moisture Condition
- 5) Primary and Secondary Structure

1) LITHOLOGY/TEXTURE

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

	Particle Size (ASTM D2487)	Relative Proportions (CFEM 4th Edition. 2006)	
Boulder	> 300 mm	Gravel, Sand, Silt, Clay	>35% and main fraction
Cobble	75 - 300 mm		
Gravel	4.75 - 75 mm	and	>35%
Coarse	19 - 75 mm		
Fine	4.75 - 19 mm	Gravelly, Sandy, Silty, Clayey	20 - 35%
Sand	0.075 - 4.75 mm		
Coarse	2 - 4.75 mm	some	10 - 20%
Medium	0.425 - 2 mm		
Fine	0.075 - 0.425 mm		
Silt and Clay	Smaller than 0.075 mm	trace	<10%

	Gradation		Particle Shape
Well Graded	Having a wide range of grain sizes and substantial amount of all intermediate sizes	Angular	Sharp edges and relatively plane sides with unpolished surfaces
Uniform or Poorly	Possessing particles of predominantly one size	Subangular	Similar to 'angular' but have rounded edges
Gap Graded	Possessing particles of two distinct sizes	Rounded	No edges and smoothly curved surfaces

Also may be flat, elongated or both

Glacial Till: Glacial till is comprised of a heterogeneous mixture of clay, silt, sand and gravel-sized particles. Due to the nature of formation and deposition, glacial till also inherently contains larger particle sizes (cobbles and boulders).

2) COLOUR AND OXIDATION

A soils colour may be described either qualitatively in the field at the soils natural moisture content using common colours (eg., light grey, light brown, dark grey, etc.), or quantitatively using the Munsell Book of Colour (eg. 5Y 3/1). The Munsell notation combines three variables, hue, value, and chroma to describe the soil colour. The hue indicates its relation to red, yellow, green, blue, and purple. The value indicates its lightness. The chroma indicates its strength of departure from a neutral of the same lightness. Quantitative determination of colour using the Munsell Book of Colours is completed after the soil has been allowed to dry at a low temperature.

Departure of the soil colour from a neutral colour indicates the soil has been oxidized. Oxidation of a soil occurs in a oxygen rich environment where most commonly metallic iron, oxidizes and turns a neutral coloured soil 'rusty' or reddish brown. Oxidized manganese gives a purplish tinge to the soil. Oxidation may occur throughout the entire soil mass or on fracture/joint/fissure surfaces. "Mottled" refers to a soil that is spotted, blotched or streaked with different shades or colours.



**Classification of Soils for Engineering Purposes
(Including Identification and Description)
As Adopted by P.F.R.A.**

Major Divisions		Group Symbols	Typical Names	Classification Criteria	
Coarse-grained soils More than 50% retained on No. 200 sieve (>0.075 mm)	Clean gravel (<5% fines)	GW	Well-graded gravels, gravel sand mixtures	Classification on basis of percentage of fines Less than 5% pass No.200 sieve... GW, GP, SW, SP More than 12% pass No. 200 sieve... GM, GC, SM, SC 5-12% pass No. 200 sieve... Borderline classifications requiring use of dual symbols	$C_u=D_{60}/D_{10}$ must be > 4; and, $C_c=(D_{30})^2/(D_{10} \times D_{60})$ between 1 and 3
		GP	Poorly-graded gravels, gravel - sand mixtures		Not meeting both above criteria for GW
	Gravel with fines (>12% fines)	GM	Silty gravel, gravel - sand - silt mixtures		Atterberg limits below "A" line or $PI < 4$
		GC	Clayey gravel, gravel - sand - clay mixtures		Atterberg limits on/above "A" line with $PI > 7$
	Clean sand (<5% fines)	SW	Well-graded sand, gravelly sands		$C_u=D_{60}/D_{10}$ must be > 6; and, $C_c=(D_{30})^2/(D_{10} \times D_{60})$ between 1 and 3
		SP	Poorly-graded sand, gravelly sands		Not meeting both above criteria for SW
	Sand with fines (>12% fines)	SM	Silty sands, sand - silt mixtures		Atterberg limits below "A" line or $PI < 4$
		SC	Clayey sands, sand - clay mixtures		Atterberg limits on/above "A" line with $PI > 7$
Fine-grained soils 50% or more passes No. 200 sieve (≤ 0.075 mm)	Inorganic	ML	Silts, silty or clayey fine sands or clayey silts with slight plasticity	<p align="center">Plasticity Chart</p> <p>The Plasticity Chart plots Plasticity Index (PI) on the y-axis (0 to 50) against Liquid Limit (LL) on the x-axis (0 to 80). A diagonal line is labeled 'A line = 0.73(LL-20)'. Vertical lines are drawn at LL=30 and LL=50. Horizontal lines are drawn at PI=7 and PI=17. Regions are labeled: ML (LL < 20, PI < 7), CL (LL < 30, 7 < PI < 17), OL (LL < 30, PI > 17), MH (LL > 30, PI < 7), CH (LL > 30, 7 < PI < 17), OH or MH (LL > 30, PI > 17), and CI (LL > 40, 7 < PI < 17). A shaded region between LL=15 and LL=25, PI=7 and PI=17 is labeled 'CL-ML'.</p>	
		CL	Clays of low plasticity		
	Organic	OL	Organic silts or clay of low plasticity		
	Inorganic	CI	Silts or clays of medium plasticity		
	Inorganic	MH	Micaceous or diatomaceous fine sandy or silty soils, elastic silts		
		CH	Silts or clays of high plasticity		
	Organic	OH	Organic clays or silts of high plasticity		
		PT	Peat, muck and other highly organic soils		Boundary classification: Soils possessing characteristics of two groups are designated by combinations of group symbols. For example GW - GC, well graded gravel - sand mixture with clay binder



3) CONSISTENCY AND CONDITION

The consistency of a cohesive soil is a qualitative description of its resistance to deformation and can be correlated with the undrained shear strength of the soil. Undrained shear strength can be estimated based on field identification methods (resistance to fist/thumb/fingernail penetration), pocket penetrometer test or standard penetration resistance (SPT N-index, ASTM D1586).

The condition of a coarse grained soil qualitatively describes the soil compactness and can be correlated with the SPT N-index.

The standard penetration resistance is described as the number of blows, N, required to drive a 51 mm outside diameter (O.D.) split barrel sampler into the soil a distance of 0.3 m (from 0.15 to 0.45 m) with a 63.5 kg weight having a free fall of 0.76 m.

a) Consistency and Plasticity of Cohesive Soils

Consistency	*SPT N-Index	Undrained Shear Strength (kPa) (CFEM 4 th Ed., 2006)	Pocket Penetrometer (kg/cm ²)	Field Identification (ASTM D2488)
Very Soft	0 – 2	<12	<0.25	Easily penetrated several cm by fist
Soft	2 – 4	12 – 25	0.25 – 0.5	Easily penetrated several cm by thumb
Firm	4 – 8	25 – 50	0.5 – 1.0	Can be penetrated several cm by the thumb with moderate effort
Stiff	8 – 15	50 – 100	1.0 – 2.0	Readily indented by thumb but penetrated only with great effort
Very Stiff	15 – 30	100 – 200	2.0 – 4.0	Readily indented by thumbnail
Hard	>30	>200	>4.0	Indented with difficulty by thumbnail

* uncorrected

The plasticity of cohesive soils is a measure of the soils ability to deform without rupture. The plasticity should be estimated in the field as low (liquid limit <30), medium (liquid limit between 30 and 50) or high (liquid limit >50). The plasticity can be confirmed in the laboratory through Atterberg limits testing.

b) Compactness Condition of Coarse Grained Soils

Compactness Condition	*SPT N-Index
Very Loose	0 – 4
Loose	4 – 10
Compact	10 – 30
Dense	30 – 50
Very Dense	>50

* uncorrected

4) MOISTURE CONDITION

Moisture Condition	Description
Dry	No moisture, dusty, dry to touch
Moist	Damp but contains no visible water
Wet	Visible, free water, usually soil is below piezometric surface

**5) PRIMARY AND SECONDARY STRUCTURE****(a) Primary Soil Structure (Depositional)****Geometry**

Stratum	A single sedimentary unit which is visibly separable from other strata by a discrete change in lithology and/or sharp physical break
Homogeneous	Uniform in nature
Stratified	Consisting of a sequence of layers which are generally different in texture or colour
Thinly Laminated	Stratified with layer thickness < 2 mm
Laminated	Stratified with layer thickness between 2 – 10 mm
Bedded	Stratified with layer thickness >10 mm
Very Thinly Bedded	Stratified with layer thickness between 10 – 50 mm
Thinly Bedded	Stratified with layer thickness between 50 – 600 mm
Thickly Bedded	Stratified with layer thickness between 600 – 1,200 mm
Massive	Stratified with layer thickness >1,200 mm
Lenses	Inclusions of small pockets of different soils

Bedding Structures

Cross bedding	Internal bedding inclined to the general bedding plane
Ripple bedding	Internal wavy bedding
Graded bedding	Internal gradation of grain size from coarse at base of bed to finer at top of bed (or vice versa)
Horizontal bedded	Internal bedding parallel and flat lying

(b) Secondary Soil Structure (Post Depositional)**Accretionary Structures** (Includes nodules, concretions, crystal deposits, veins, colour banding)

Cementation	Chemically precipitated material (commonly calcite) which binds soil grains together
Salt Crystals	Groundwater flowing through soils often precipitates residual salts in the soil structure. Common salt precipitates are gypsum, calcite and glauber salts.

Fracture Structures

Fracture	A break or discontinuity in the soil caused by stress exceeding the material strength
Joint	A fracture along which no displacement has occurred
Blocky	A cohesive soil that can be broken down into small pieces which resist further breakdown, typically resulting from soil fracture patterns
Slickensided	Fractures that are slick and glossy (plane of weakness), typically associated with shear displacement of soils
Brecciated	Containing randomly oriented angular fragments in a finer mass, typically associated with shear displacement of soils



Symbols Used on Borehole Logs

	WATER		CLAY AND SILT		OXIDIZED TILL
	ICE		SAND AND SILT		UNOXIDIZED TILL
	ASPHALT		SAND, SANDSTONE		CALCAREOUS SHALE OR LIMESTONE
	FILL		GRAVEL		COAL
	TOPSOIL OR ORGANICS		SAND AND GRAVEL		SALT
	CLAY, SHALE, CLAYSTONE, MUDSTONE		WASTE ROCK		SILT, SILTSTONE
	BEDROCK/GRANITE		PEAT		COBBLES/BOULDERS

Sampling

	CORE		SPLIT SPOON		BAGGED / GRAB / DISTURBED		SHELBY
---	------	---	-------------	---	---------------------------	---	--------

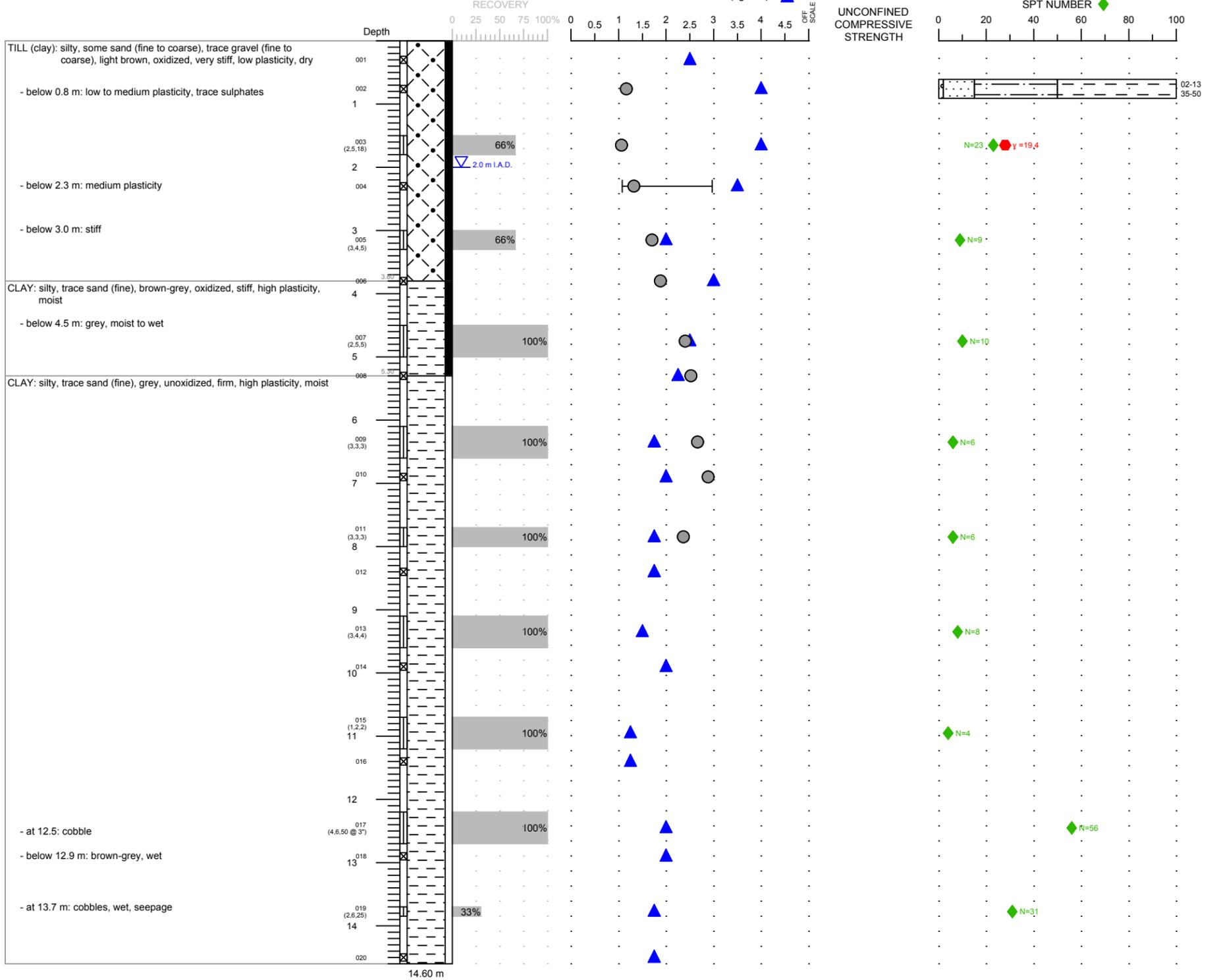
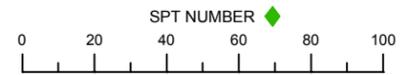
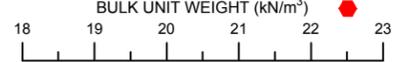
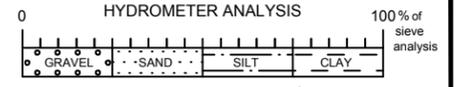
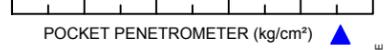
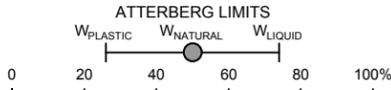
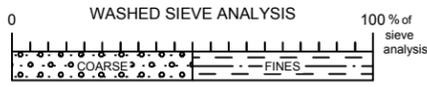
Soil Oxidation

	OXIDIZED ZONE		UNOXIDIZED ZONE
---	---------------	---	-----------------

Piezometer and Borehole Completion Symbols

	GROUT		BENTONITE CHIPS		CUTTINGS
	SAND PACK		SLOUGH		WATER LEVEL (I.A.D.)
					WATER LEVEL DATE

BOREHOLE 661472-01
CITY OF WINNIPEG - MCPHILLIPS PUMPING STATION
2019
 5530779 N 631291 E
 NAD 83 ZONE 14



REFERENCE DRAWINGS

NOTES

- Groundwater at 2.0 m immediately after drilling (I.A.D).
- SPT refusal due to cobbles at 12.6 m.
- Auger refusal at 14.6 m.
- Standard Penetration Tests (SPT) conducted with 63.5 kg (140 lb) automatic trip hammer falling 762 mm (30 inches).
- (#, #, #) denotes SPT blows per 152 mm (6.0 inches).
- Depths are in metres (m).
- Borehole coordinates are from Google Earth and are approximate.



CLIENT CITY OF WINNIPEG	PROJECT LOCATION MCPHILLIPS PUMPING STATION, WINNIPEG, AB
-----------------------------------	--

APPROVED BY	
DRAWN BY	E. OVCINA
PROJECT No.	661472
SCALE	1:75
DATE	2019 10 02

LIMITATION
 This drill log is a summary of the conditions estimated by the field personnel at the specific location at the time of drilling. The conditions and properties described above will vary between locations and may vary with time.

DRAFT

DWG No	DESCRIPTION	SUPERVISOR	K. RUTHERFORD
CONTRACTOR	MAPLE LEAF DRILLING	LOGGED BY	K. RUTHERFORD
OPERATOR	PRESTON / JESSY	DRILLED DATE	2019 09 27
TYPE OF DRILL RIG	TRUCK MOUNTED MOBILE B40	INSTALLATION DATE	N/A
ABANDONMENT	AUGER CUTTINGS		

Appendix III

Laboratory Testing Results

ATTERBERG LIMITS TEST REPORT



(Test Reference: ASTM D 4318)



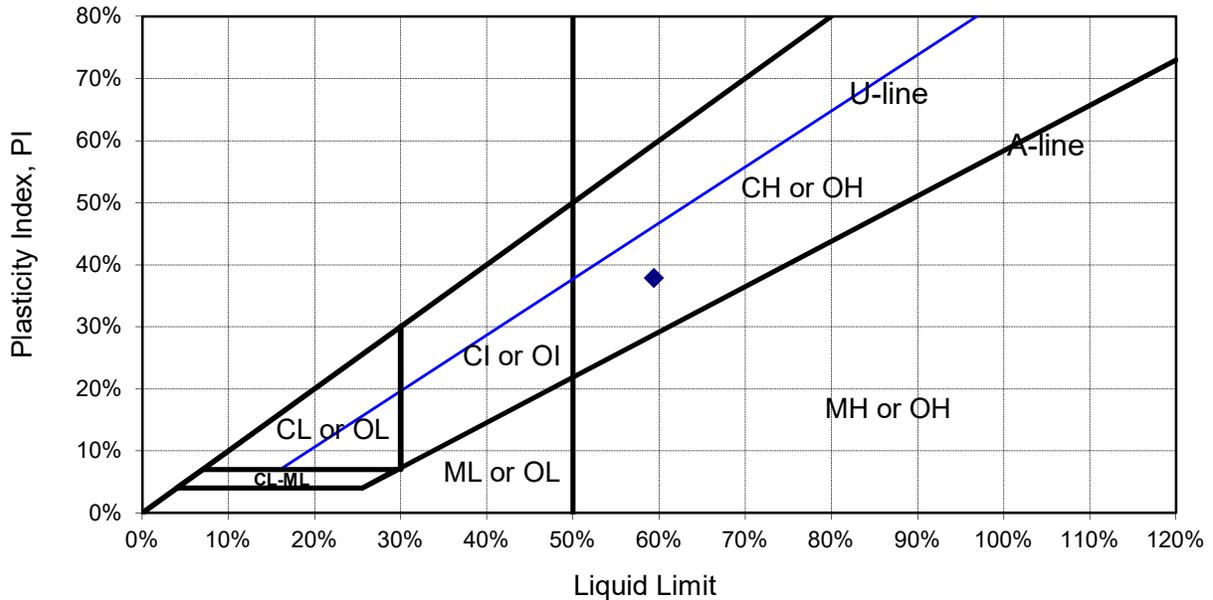
Sample: 004 at 2.3m (air-dried)

SNC • LAVALIN

PLASTIC LIMIT		LIQUID LIMIT (METHOD B)		
		# of Blows	21	23
Tare Wt, g	14.26	Tare Wt, g	14.59	14.16
Wet + Tare, g	26.05	Wet + tare, g	32.92	32.44
Dry + Tare, g	23.96	Dry + tare, g	26.01	25.57
M%	21.5%	Water content	60.5%	60.2%
		Adjusted W/C	59.2%	59.6%
				AVERAGE
				59.4%

SUMMARY	COMMENTS
Plastic Limit: 21.5%	-
Liquid Limit: 59.4%	
Plasticity Index: 37.8%	
Classification: CH	

Natural Water Content: **26.4%**

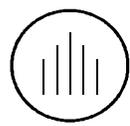


Checker: *Don Hugel*

Reviewer: *[Signature]*

The testing services reported here have been performed in accordance with accepted local industry standards. The results presented are for the sole use of the designated client only. This report constitutes a testing service only. It does not represent any interpretation or opinion regarding specification compliance or material suitability. Engineering interpretation will be provided by SNC Lavalin upon request.

Client: City of Winnipeg
 Project: McPhillips Pumping Station
 Project #: 626988
 Date: 3-Oct-2019



HYDROMETER TEST REPORT



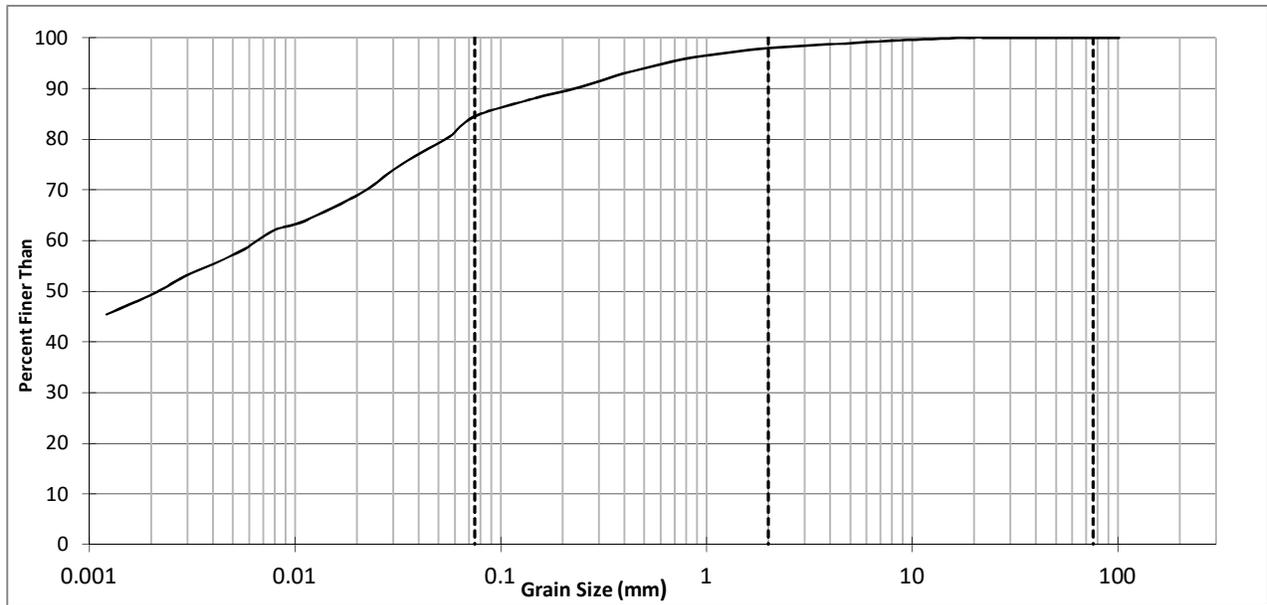
(Test Reference: ASTM D7928)



SNC • LAVALIN

Sample: 002 - at 0.8m

Mechanical Analysis			Hydrometer Analysis		Summary of Analysis	
Sieve	Dia. (mm)	% Finer	Dia. (mm)	% Finer	Particle Size Distribution Summary	
4"	101.6	100	0.0563	80.4	% Cobble	0
3"	76.2	100	0.0405	77.2	% Gravel	2
2"	50.8	100	0.0291	73.5	% Sand	13
1"	25.4	100	0.0210	69.3	% Silt Size (<75μ>2μ)	35.3
3/4"	19.1	100	0.0111	63.8	% Clay Size (<2μ)	49.7
3/8"	9.5	100	0.0079	62.0	Dispersing Agent used:	
#4	4.75	99	0.0057	58.3	Sodium Hexametaphosphate	
#10	2	98	0.0041	55.5		
#20	0.85	96	0.0029	52.9		
#40	0.425	93	0.0021	49.6		
#60	0.25	90	0.0012	45.4		
#100	0.15	88				
#200	0.075	85				

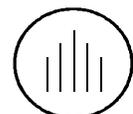


FINES (silt, clay) 85 %	SAND 13 %	GRAVEL 2 %	COBBLE 0 %
-------------------------	-----------	------------	------------

Checker: *Don McPhillips* Reviewer: *[Signature]*

The testing services reported here have been performed in accordance with accepted local industry standards. The results presented are for the sole use of the designated client only. This report constitutes a testing service only. It does not represent any interpretation or opinion regarding specification compliance or material suitability. Engineering interpretation will be provided by SNC Lavalin upon request

Client: Project City of Winnipeg
 Project #: McPhillips Pumping Station
 Date: 661472
 2019-07-11





SNC • LAVALIN

216 - 1st Avenue South
Saskatoon, Saskatchewan, Canada S7K 1K3
306.668.6800

www.snclavalin.com

