FINAL REPORT

GEOTECHNICAL INVESTIGATION PROPOSED PUMPING STATION ADDITION 5719 ROBLIN BLVD WINNIPEG, MANITOBA

Prepared for: City of Winnipeg

SNC-LAVALIN INC.

June 19, 2017
FINAL REPORT
Project No.: 644548

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1 INTRODUCTION

This report presents the results of the geotechnical investigation conducted by SNC-Lavalin for the proposed City of Winnipeg Pumping Station expansion located at 5719 Roblin Blvd (north of the intersection of Roblin Blvd and Community Row) in Winnipeg, Manitoba. The geotechnical investigation included the field investigation program (borehole drilling), field and laboratory soil testing and a report that provides geotechnical foundation design recommendations for development of the site.

The proposed pump house expansion covers an area of approximately 28 m^2. The purpose of the addition is to house electrical equipment with a total weight of approximately 2,500 kg. It is understood that the expansion will consist of an at-grade concrete structure with a grade-supported concrete floor slab. It is anticipated that a pile/grade beam foundation system will be utilized to support the addition.

2 SITE LOCATION AND DESCRIPTION

The existing pumping station is located at 5719 Roblin Blvd in Winnipeg, Manitoba. The proposed addition will be constructed along the north edge of the existing building structure. The site is currently undeveloped land with grass as the primary vegetation. A site plan showing the existing pumping station and proposed addition and the location of the borehole has been shown as Figure I.1, Appendix I.

3 SCOPE OF WORK

The objective of the geotechnical investigation was to conduct a field investigation within the proposed building addition footprint and to provide geotechnical recommendations to support design of the proposed structure foundations. The following scope of work was completed:

- Field investigation consisting of one (1) deep borehole, geotechnical field tests, logging of soils and collection of soil samples for laboratory testing. The deep borehole was drilled to a depth of 9.5 m below ground level (mbgl);
- Laboratory testing of select soil samples obtained from the borehole, including water contents, Atterberg limits, grain size distribution analysis, and unconfined compressive strengths (UCS); and,
- Preparation of a report summarizing the field investigation and providing geotechnical recommendations containing foundation recommendations.

The borehole location is shown on the site plan in Appendix I, and the borehole log has been included in Appendix II.

4 GEOTECHNICAL INVESTIGATION DETAILS

4.1 Drilling Investigation

The field investigation was conducted on April 21, 2017. Maple Leaf Drilling Ltd. from Winnipeg, Manitoba utilized a track-mounted drill rig equipped with continuous flight, solid stem augers and an automatic standard penetration test (SPT) hammer to drill the borehole.

4.1.1 Borehole Drilling

Disturbed soil samples were collected from the auger cuttings and from the SPT sampler. Relatively undisturbed Shelby tube samples were taken at select locations. All soil samples were transported to the SNC-Lavalin soil testing laboratory Saskatoon, Saskatchewan. The soil samples were stored in a humidity-controlled room to prevent drying prior to testing. Soil samples collected from the borehole are shown on the borehole logs in Appendix II.

Field testing included pocket penetrometer tests (PP's) conducted on all cohesive samples collected and SPT's conducted at selected depths. The results of field tests are presented on the borehole log 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 bentonite chips to surface.

4.2 Geotechnical Laboratory Testing

Geotechnical laboratory tests were conducted on soil samples obtained from the borehole. The laboratory analyses included water contents, Atterberg limits, grain size distribution analysis, and unconfined compressive strength (UCS) tests.

The detailed laboratory test results are provided in Appendix III and also annotated on the borehole log presented in Appendix II.

5 SUBSURFACE CONDITIONS

5.1 Soil Profile

The general subsurface soil conditions consisted of a thin veneer of organic topsoil at the surface. The organic topsoil was underlain by a 0.6 m thick layer of clay fill, followed by 7.6 m thick layer of high plastic clay which was underlain by glacial till which extended to a depth of at least 9.5 mbgl, the maximum depth drilled.

The clay fill was silty, contained trace amounts of fine to coarse grained sand, trace amounts of organics, occasional rootlets, was mottled brown to black, firm, medium plastic and moist. The clay contained trace silt, was brown, firm becoming stiff with depth, high plastic, moist and contained occasional silt pockets and oxidation. The glacial silt till was clayey, contained some fine to coarse grained sand, trace fine to coarse grained gravel, was light brown, dense becoming very dense with depth, non-plastic, and moist.

5.2 Groundwater Seepage and Sloughing

No groundwater seepage or sloughing was encountered during the drilling and within 10 minutes after the drilling was completed.

5.3 Cobbles and Boulders

Suspected cobbles/boulders were encountered within the glacial silt till deposit underlying the high plastic clay deposts during test drilling.

Glacial till is comprised of a heterogeneous mixture of clay, silt, sand and gravel-sized particles. Due to the nature of the formation and deposition, glacial till also inherently contains larger particle sizes (cobbles and boulders). Cobbles and boulders are often located randomly within glacial till deposits but can also form sorted layers, such as boulder pavements. The actual location and frequency of cobbles and boulders varies and the probability of encountered 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.

6 GEOTECHNICAL RECOMMENDATIONS

6.1 Geotechnical Considerations

It is understood that the building will be supported on a pile/grade beam foundation system with a grade-supported concrete floor slab. It is also understood that the floor elevation of the building will be situated near the existing ground level.

The subsurface soil profile consisted of 0.6 m of clay fill overlying high plastic clay, followed by glacial till. No groundwater seepage/sloughing were encountered within the borehole and the borehole remained open during and immediately after drilling.

A deep foundation system consisting of drilled, cast-in-place concrete piles should perform satisfactorily at this site.

Design recommendations have been presented for site preparation; fill materials, placement and compaction; foundations; grade-supported concrete slabs; and, foundations concrete.

6.2 Site Preparation

6.2.1 General

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 SNC-Lavalin personnel to verify the removal of unsuitable materials and to provide additional recommendations, as appropriate. Unsuitable materials include topsoil, organic matter, and clay fill material. 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 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 200 mm, moisture conditioned (wetted or dried) to within ± 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.

6.2.2 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 should be over excavated to expose more competent soil and replaced with suitable engineered fill. Alternately, the use of geosynthetics (woven geotextile, geogrid in conjunction with non woven geotextile, or, combination geotextile/geogrid products), possibly in conjunction with some over excavation, may be an alternative.

If geosynthetics are utilized, it is recommended that granular fill materials be placed directly over the geosynthetics. The geosynthetics should be placed in accordance with the manufacturer's recommendations. Construction techniques should be designed to minimize the potential for damage to the geosynthetics and underlying subgrade soils (ie, end-dump and spread methods, use of long reach and/or low contact pressure equipment, etc). SNC-Lavalin should be retained to provide guidance with respect to subgrade improvement measures.

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 Fill Materials, Placement and Compaction

6.3.1 General

All proposed fill material should comply with the recommendations provided in this report and should be approved by SNC-Lavalin 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.

6.3.2 Subgrade Fill

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

6.3.3 Structural Fill

Well-graded granular material 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).

All structural fill should be placed in thin lifts (150 mm thickness, maximum), moisture conditioned (wetted or dried) to within ± 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 gradation requirements for base course and sub-base course material have been presented in Table 6.1. Alternate gradations may be acceptable but should be approved by SNC-Lavalin 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 (ie, greater than 50 mm) removed.

Table 6.1 – Base and sub base gradation specifications.

Sieve Size		Percent Passing by Weight	
		Base Course Type 33	Sub-Base Type 6
50	mm		100
18	mm	100	
12.5	mm	75 -100	
5	mm	50 - 75	
2	mm	32 - 52	0 - 80
900	µm	20 - 35	
400	µm	15 - 25	0 - 45
160	µm	8 - 15	0 - 20
71	µm	6 - 11	0 - 6
Plasticity Index		0 - 6	0 - 6
Fractured Face %		Min 50	
Lightweight pieces %		Max 5	

6.3.4 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 (ie, 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. In areas where there will be no surface cover (asphalt, concrete, etc), it is recommended that the excavations be capped with low hydraulic conductivity soils to limit surface water ingress into the utility trench. In areas where there will be a permeable surface cover (ie, graveled areas, landscape areas, etc). The drainage adjacent to the utility trench should provide for positive drainage away from the trench.

6.3.5 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 (ie, 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 6.2.

Table 6.2 – Estimated fill settlement versus compaction level.

Compaction Level (%SPMMD)	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

SPMDD = Standard Proctor Maximum Dry Density (± 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 6.2 if frozen fill materials are utilized (particularly for cohesive fill materials).

6.4 *Foundations*

6.4.1 Limit States Design

6.4.1.1 *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:

- Exceed the load carrying ability of the ground that supports the foundation (ie, 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 (ie, serviceability) of the structure. For foundation design, SLS generally consist of:

- Excessive movements (eg, 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.

6.4.1.2 *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 2010 (NBCC). The recommended geotechnical resistance factors (ϕ) as per the National Building Code of Canada 2010 (NBCC) are as follows:

1. Deep Foundations
   (a) Resistance to axial load
      (i) Semi empirical analysis using laboratory and in situ test data (ϕ = 0.4)
      (ii) Analysis using static loading test results (ϕ = 0.6)
      (i) Analysis using dynamic monitoring results (ϕ = 0.5)
      (ii) Uplift resistance by semi empirical analysis (ϕ = 0.3)
      (iii) Uplift resistance using loading test results (ϕ = 0.4)
   (b) Horizontal load resistance (ϕ = 0.5)

Ultimate geotechnical resistances to axial loads for deep foundations were calculated using semi empirical analysis using laboratory and in situ test data.

6.4.2 Drilled, Cast-in-Place Concrete Piles

Conventional piling equipment utilized in western Canada for typical straight shaft drilled piles is not meant to clean the base of the pile hole. As such, drilled straight shaft, cast in place concrete piles may be designed on the basis of shaft resistance only.

The shaft resistance values of the subgrade soils are presented in Table 6.3.

Table 6.3 – Shaft resistance (drilled piles).

Depth (mbgl)	ULS Shaft Resistance (kPa)	SLS Shaft Resistance (kPa)	
		Compression	Tension
0 to 2	0	0	0
2 to 8.5	35	14	11
8.5 to 9.5	125	50	38

The following recommendations should be considered in the design of drilled, cast-in-place concrete piles:

1. To minimize frost heave potential, drilled straight shaft concrete piles should be extended to a minimum depth of 6 m. The potential for frost heave of the piles can be reduced by using a sonotube form for the uppermost 2 m (below ground surface) of the pile shaft. The diameter of the sonotube form should be a minimum of 50 mm smaller than the diameter of the drilled hole. It is noted that the use of a sonotube form may not be practical for piles subject to significant lateral loads, as the sonotube portion of the pile will not provide any lateral capacity.
2. Pile reinforcement must be adequate to withstand all vertical, lateral and tensile forces within the pile.
3. A minimum pile diameter of 400 mm is recommended.
4. A minimum centre-to-centre pile spacing of three pile diameters is recommended.
5. Although not anticipated, if groundwater seepage or sloughing conditions are encountered during construction, casing will be required to maintain the pile holes open and dry for placement of the reinforcing steel and concrete. The annular space between the casing and drilled hole must be filled with concrete. As the casing is extracted, concrete in the casing must have adequate head to displace all water in the annular space. Water which accumulates on top of the pile upon removal of the casing must be removed to ensure the integrity of the concrete is not compromised.
6. Pile holes should be filled with concrete as soon as possible after drilling. Excess water should not be allowed to collect within the drilled hole (if applicable). If excess water collects in the drilled hole, it will be necessary to remove the water (by pumping or bailing) prior to placing reinforcing steel and concrete. Vibration of the concrete in the upper 3 m of the pile shaft is required to produce uniform strength concrete.
7. Concrete shall be fed to the bottom of the drilled shaft by pumping and filled from bottom up or, using the free fall method or, another method approved by the structural engineer. If the free fall method is used, the concrete must be poured through a centering chute, making it fall down at the centre of the hole, and minimize the fresh concrete hitting the reinforcing steel or the side of the shaft.
8. Continuous monitoring by SNC-Lavalin during pile installation is recommended to document the installation of each drilled, straight shaft concrete pile installed at this site.

6.4.3 Pile Settlement and Pile Group Effects

For individual drilled piles, pile settlement is not expected to exceed 10 mm. Differential settlement between piles is anticipated to be about half the total settlement of a single pile. Pile group settlement will be larger than for individual piles, and will depend on the pile group size/geometry, pile type, pile depth, etc. Estimates of pile group settlement can be provided once the pile group configurations and loads have been finalized.

If pile groups are required to achieve the required structural capacity, the minimum centre to centre pile spacing should be three times the pile diameter. A reduction in pile capacity will not be required for pile groups where the centre to centre pile spacing is at least three pile diameters. For pile groups with a centre to centre spacing of less than three pile diameters, a capacity reduction may need to be applied. In this case, SNC-Lavalin should be contacted to reassess the pile group capacity (will be affected by the number of piles, the pile layout and pile diameters).

6.4.4 Grade Beams and Pile Caps

Grade beams should be constructed to allow for a minimum of 100 mm of net void space between the underside of the grade beam and the subgrade soil (compressible void form). The finished grade adjacent to each grade beam should be capped with hard surfacing or well compacted, low permeable material and should be positively drained away from the grade beam so that surface runoff is not allowed to infiltrate and collect in the void space. If water is allowed to accumulate in the void space, the beneficial effect will be negated and frost heaving may occur.

Exterior pile caps exposed to freezing conditions should be based below the potential depth of frost penetration or protected against frost action. Pile caps based above the frost penetration depth should be constructed to allow for a minimum of 100 mm of net void space between the underside of the pile cap and the subgrade soil (compressible void form). As with grade beams, the finished grade adjacent the pile cap should be positively drained away from the pile cap so that surface runoff is not allowed to infiltrate and collect in the void space. Alternatively, the pile caps may be protected from frost action by strategically located, rigid polystyrene insulation. Further insulation recommendations can be provided upon request.

The use of bond breakers between the foundation and the soil can reduce the potential for foundation movements due to adfreezing forces, and is recommended.

6.5 Grade Supported Concrete Slabs

The near surface subgrade soils consisted of clay fill overlying high plasticity clay. Grade-supported concrete slabs should perform satisfactorily provided that some floor slab movements/cracking can be tolerated. If some differential movements/floor cracking cannot be tolerated, then a structural slab should be constructed.

For continually heated areas, the following recommendations should be incorporated into the design of reinforced, grade supported, cast in place concrete slabs at this site:

1. Over-excavate, as required, to allow for a minimum granular fill thickness of 450 mm below the floor slab. The uppermost 150 mm of fill immediately below the slab should consist of crushed granular base course material.
2. All structural fill should be placed in thin lifts (150 mm thickness, maximum), moisture conditioned (wetted or dried) to within ± 2% of optimum moisture content, and uniformly compacted to at least 100% of SPMDD tested in accordance with ASTM Method D 698.
3. Separation joints should be used to isolate the slab from foundation walls, columns, etc.
4. Reinforce the concrete slab and provide control joints at regular intervals to provide for controlled cracking.
5. The finished grade should be landscaped to provide for positive site drainage away from the structure.
6. Concrete slabs should not be constructed on loose, softened, desiccated, frozen or wet soil.
7. Frost should not be allowed to penetrate beneath the concrete slab just prior to, during or after construction.
8. Continuous quality control inspection by SNC-Lavalin should be provided during fill placement.

6.6 Foundation Concrete

The clay soils in the Winnipeg area contain sulphates that will cause deterioration of concrete. The class of exposure for concrete in contact with clay soil in the Winnipeg area is considered to be severe (S-2 in CSA A23.1-09 Table 3). The requirements for concrete exposed to severe sulphate attack are provided in the following table:

Table 6.4 – Foundation Concrete Requirements

Parameter	Design Requirements
Class of exposure	S-2
Compressive strength	32 MPa at 56 days
Air content	4 to 7%
Water-to-cementing materials ratio	0.45 max.
Cement	Type HS or HSb

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. 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 installation; 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 +1.204.786.8080.

Submitted by:

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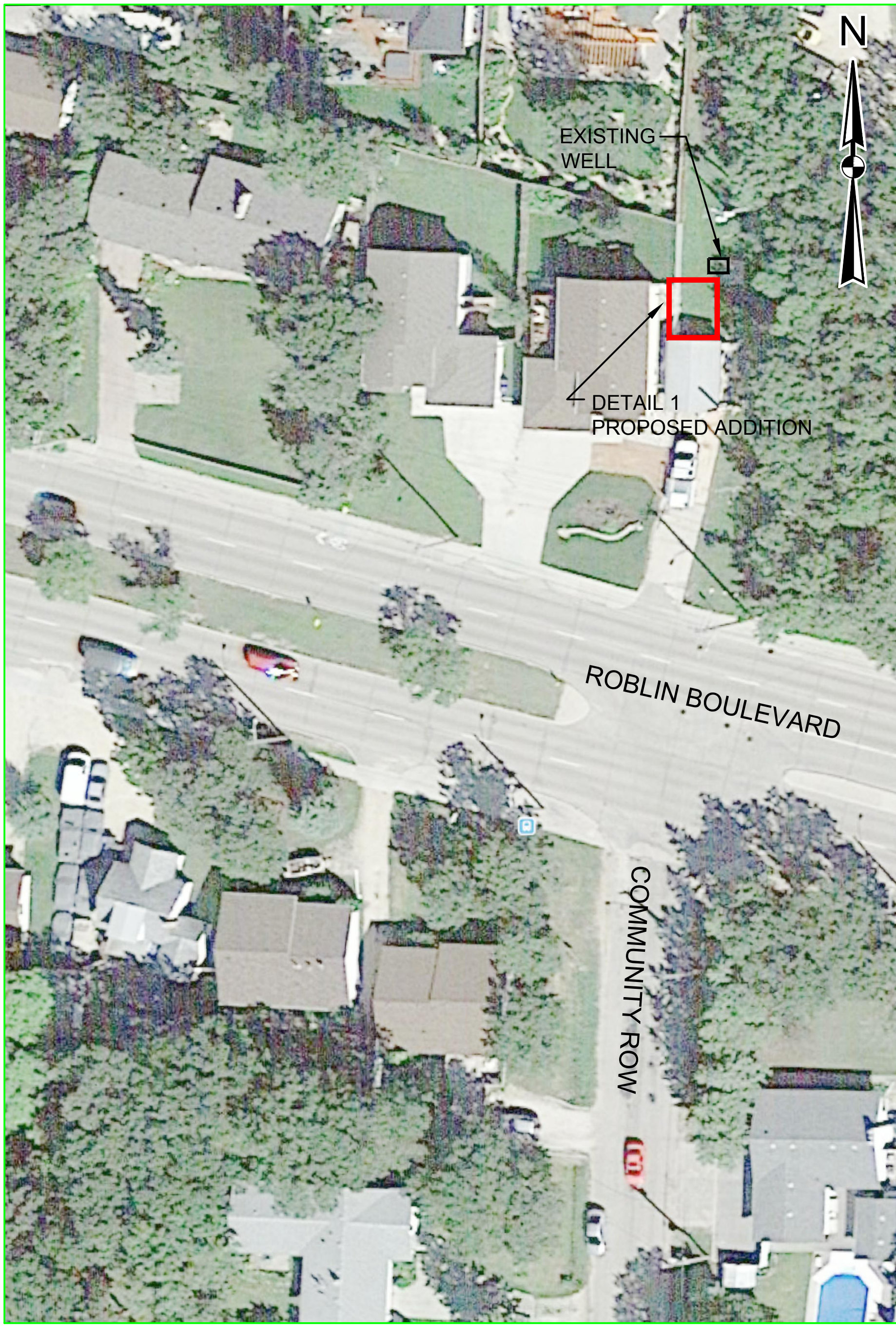
APPENDIX I

Site Plan – Borehole Location



LOCATION PLAN
SCALE 1:750

DETAIL 1
SITE PLAN
SCALE 1:100

LEGEND

● BORHOLE

REFERENCE DRAWINGS

DWG No	DESCRIPTION
BH-644548-1	BOREHOLE GEOLOGY

REVISIONS

REV	DATE	DESCRIPTION	DRN	APP

SNC·LAVALIN

CLIENT: CITY OF WINNIPEG	PROJECT LOCATION: 5719 ROBLIN BOULEVARD

TITLE
SITE PLAN-
BORHOLE LOCATION

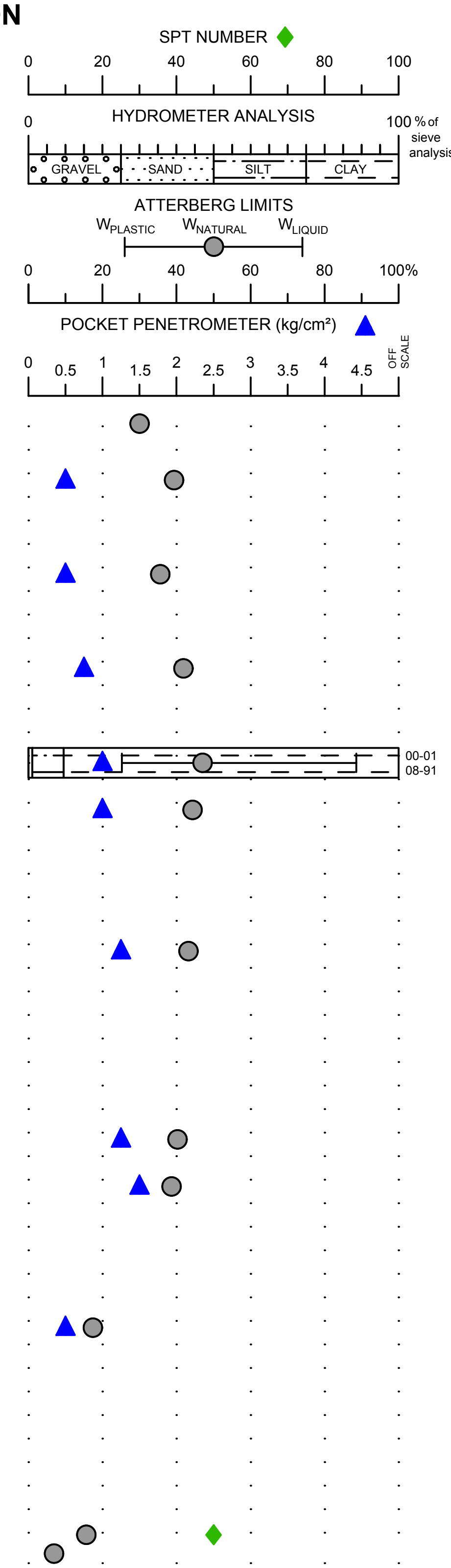
DES BY	JP	DRN BY	PB	DATE	2017 06 19	FIG No I-1	REV
CHK BY	JP	APP BY	JP	DWG No	644548-61-1		8.5X11

G:\CADD_Data\city Of Winnipeg\644548\644548-61-1.dwg

APPENDIX II

Terms and Symbols, Borehole Logs

BOREHOLE 644548-01
CITY OF WINNIPEG
COMMUNITY ROW PUMPING STATION
2017



Description	Sample	Depth (m)	Unconfined Compressive Strength
Clay (Fill): silty, trace fine to coarse grained sand, trace organics, occasional rootlets, mottled brown to black, firm, medium plastic, moist	644548-01		
		0.61	
Clay: trace silt, brown, firm, high plastic, moist, occasinal silt inclusions	644548-02		
		1	
	644548-03		
		2	
	644548-04		
- trace oxidation below 2.3 m			
- stiff below 2.9 m	644548-05	3	
	644548-06 SHELBY		64
		4	
	644548-07		
		5	
	644548-08	6	
	644548-09 SHELBY		108
- occasional glacial silt till pockets below 7.0 m		7	
	644548-10		
		8	
		8.20	
Glacial Silt Till: clayey, some finet to coarse grained sand, trace fine to coarse grained gravel, light brown, dense, non plastic, moist			
		9	
- very dense, damp below 9.1 m	644548-11 (50/125 mm)		
	644548-12		
		9.45 m	

NOTES

1. Borehole open and dry immediately after drilling (I.A.D.).
2. Standard Penetration Tests (SPT) conducted with 63.5 kg (140 lb) automatic trip hammer falling 762 mm (30 inches).
3. (#,#,#) denotes SPT blows per 152 mm (6.0 inches).
4. Depths are in metres (m).
5. Borehole terminated due to auger refusal in very dense glacial silt at 9.5 m below existing ground level.

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CLIENT	PROJECT LOCATION
CITY OF WINNIPEG	5719 ROBLIN BOULEVARD

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

CONTRACTOR	MAPLE LEAF DRILLING Ltd.	**SUPERVISOR**	J. PLOHMAN, P. Eng.	**APPROVED BY**	J. PLOHMAN
OPERATOR	N/A	**LOGGED BY**	J. PLOHMAN, P. Eng.	**DRAWN BY**	P. BIRNIE
DRILL RIG TYPE	SOLID STEM AUGER	**DATE DRILLED**	2017 04 21	**PROJECT No.**	644548
ABANDONMENT	CUTTINGS AND BENTONITE CHIPS	**DATE INSTALLED**	N/A	**SCALE**	1:60 **DATE** 2017 06 19

G:\CADD_Data\city Of Winnipeg\644548\BH-644548-01.dwg

APPENDIX III

Laboratory Testing Results

WATER CONTENT TEST REPORT

Test Reference: ASTM D2216-05)

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Client:	City of Winnipeg
Project:	Community Row Pumping Station
Project #:	644548
Technician:	DH/CH
Date:	01-May-17

Sample #	1	2	3	4	5	6
Test Hole #	TH01	TH01	TH01	TH01	TH01	TH01
Depth (m)	0.15	0.76	1.5	2.3	3.1	3.1-3.7
Tare #						
Tare Mass (g)	41.68	38.17	38.48	40.25	38.32	60.58
Wet sample + tare (g)	190.97	177.01	204.44	163.56	100.70	158.81
Dry sample + tare (g)	156.53	137.83	160.95	127.19	80.77	128.67
Wt. Dry sample (g)	114.85	99.66	122.47	86.94	42.45	68.09
Water Content (%)	29.99	39.31	35.51	41.83	46.95	44.26
Sample #	7	8	9	10	11	12
Test Hole #	TH01	TH01	TH01	TH01	TH01	TH01
Depth (m)	4.6	6.1	6.1-6.7	7.6	9.1-9.3	9.3-9.5
Tare #						
Tare Mass (g)	38.63	38.96	48.97	37.17	37.94	38.49
Wet sample + tare (g)	171.78	169.87	191.66	181.78	180.65	128.67
Dry sample + tare (g)	131.65	132.34	151.90	160.45	161.45	122.91
Wt. Dry sample (g)	93.02	93.38	102.93	123.28	123.51	84.42
Water Content (%)	43.14	40.19	38.63	17.30	15.55	6.82
Sample #						
Test Hole #						
Depth (m)						
Tare #						
Tare Mass (g)						
Wet sample + tare (g)						
Dry sample + tare (g)						
Wt. Dry sample (g)						
Water Content (%)						

Comments:

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Engineering interpretation will be provided by SNC Lavalin upon request.

ATTERBERG LIMITS TEST REPORT

(Test Reference: ASTM D 4318)

SNC·LAVALIN

Client:	City of Winnipeg
Project	Community Row Pumping Station
Project #:	644548
Technician:	JA
Date:	4-May-2017

Sample: TH01-5 at 3.1m (air-dried)

Percentage of sample retained on 425-um (No. 40) sieve: N/A

Plastic Limit

Tare #			
Tare Wt, g	14.14		
Wet + Tare, g	17.27		
Dry + Tare, g	16.64		
M%	25.2%		

Liquid Limit (method B)

# of Blows	20	22	
Tare Wt, g	14.49	14.29	
Wet + tare, g	26.89	26.22	
Dry + tare, g	20.96	20.59	
Water content	91.7%	89.4%	AVERAGE
Adjusted W/C	89.1%	87.9%	88.5%

SUMMARY

Plastic Limit:	25.2%
Liquid Limit:	88.5%
Plasticity Index:	63.3%
Classification:	CH

Natural Water Content: 47.0%

Comments:

-

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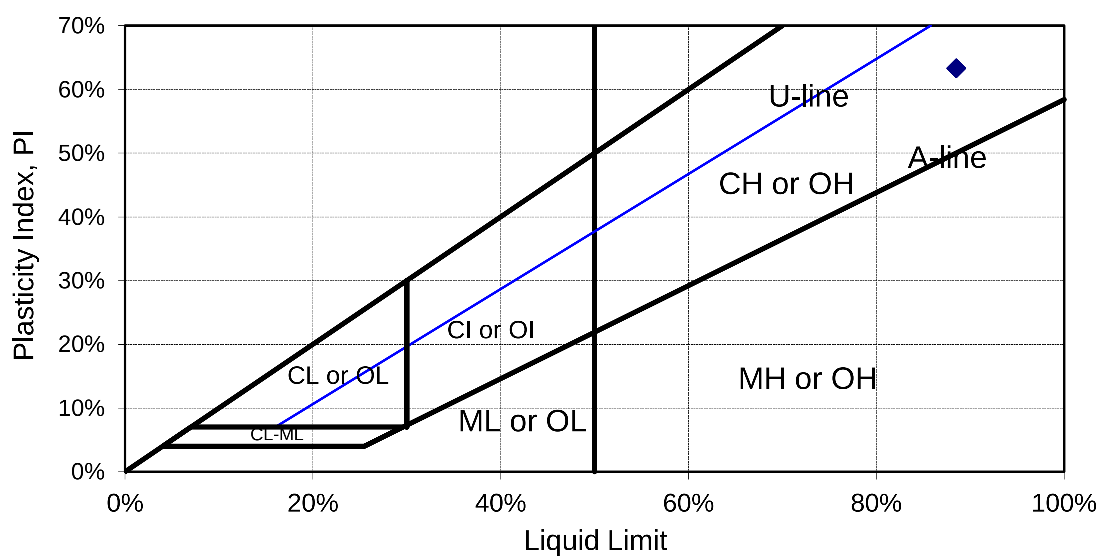

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PARTICLE-SIZE ANALYSIS REPORT

(Test Reference: ASTM D 422)

SNC·LAVALIN

Client:	City of Winnipeg
Project:	Community Row Pumping Station
Project #:	644548
Sample:	TH01-5 3.1m
Date:	04-May-17

Sieve Analysis

Sieve	Diameter (mm)	% Finer
3"	76.2	100
2"	50.8	100
1"	25.4	100
3/4"	19.1	100
3/8"	9.5	100
# 4	4.75	100
# 10	2.00	100
# 20	0.850	100
# 40	0.425	100
# 60	0.250	99
# 100	0.150	99
# 200	0.075	99

Hydrometer Analysis

Dispersing agent: *Sodium Hexametaphosphate*

Dosage of dispersing agent: *40 g/L*

Diameter (mm)	% Finer
0.0537	98.4
0.0380	98.1
0.0269	97.6
0.0191	97.1
0.0131	96.7
0.0100	94.6
0.0071	92.6
0.0051	90.5
0.0036	88.9
0.0025	85.8
0.0020	77.7
0.0012	74.0

Particle Size Distribution Summary

% GRAVEL	
% SAND	1
% SILT SIZE (<75μm>5μm)	8.5
% CLAY SIZE (<5μm)	90.5

Comments:

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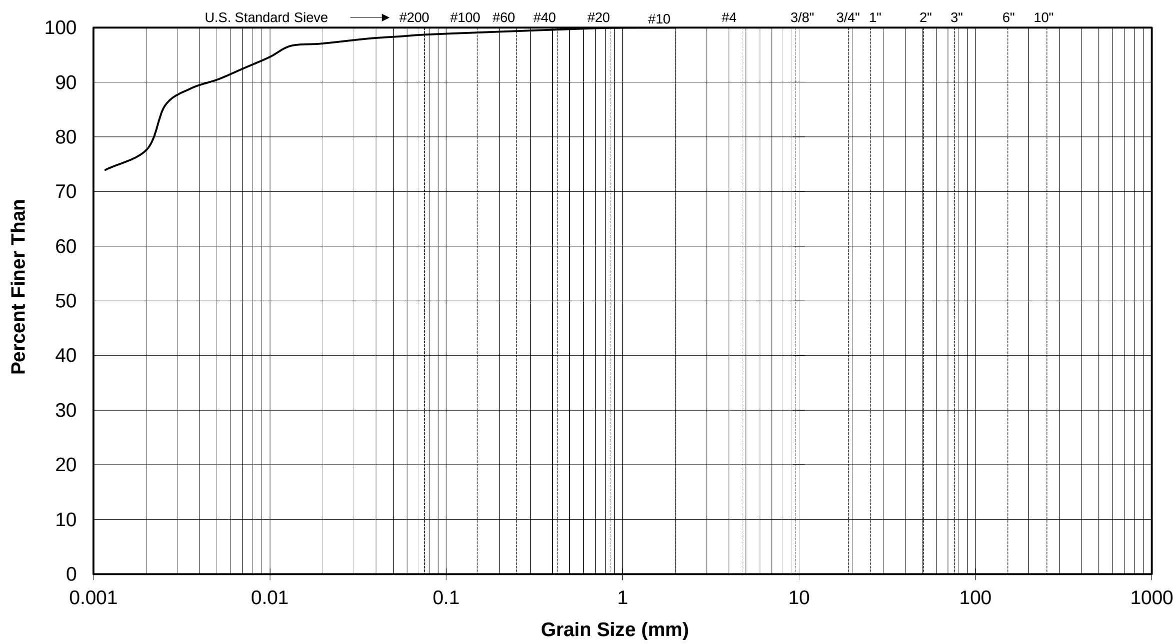

FINES (SILT, CLAY)	SAND			GRAVEL		COBBLES	BOULDERS
	Fine	Medium	Coarse	Fine	Coarse		

Unified Soil Classification System

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Engineering interpretation will be provided by SNC-Lavalin upon request.

UNCONFINED COMPRESSION TEST REPORT

(Test Reference: ASTM D2166)

SNC·LAVALIN

Client:	City of Winnipeg
Project:	Community Row Pumping Station
Project #:	644548
Technician:	CH
Checked by:	DH
Date:	May 1, 2017

Sample: TH01-06 at 3.1-3.7m

Specimen Data

Tare No:	=	L2R
Weight of Tare, g	=	60.58
Weight of Specimen Wet + Tare, g	=	158.81
Weight of Specimen Dry + Tare, g	=	127.68
Water Content, %	=	46.4%

Average Pocket Pen Result	=	0.9	
Mass of Test Specimen, g	=	1007.57	
Wet Density, kg/m^3	=	1713	
Dry Density, kg/m^3	=	1170	
Specific Gravity	=	2.70	(assumed)
Degree of Saturation	=	0.96	

Initial Diameter, D_o, cm	=	7.21
Initial Area, A_o, cm^2	=	40.84
Initial Height, L_o, cm	=	14.41
Initial Volume, V_o, cm^3	=	588.36

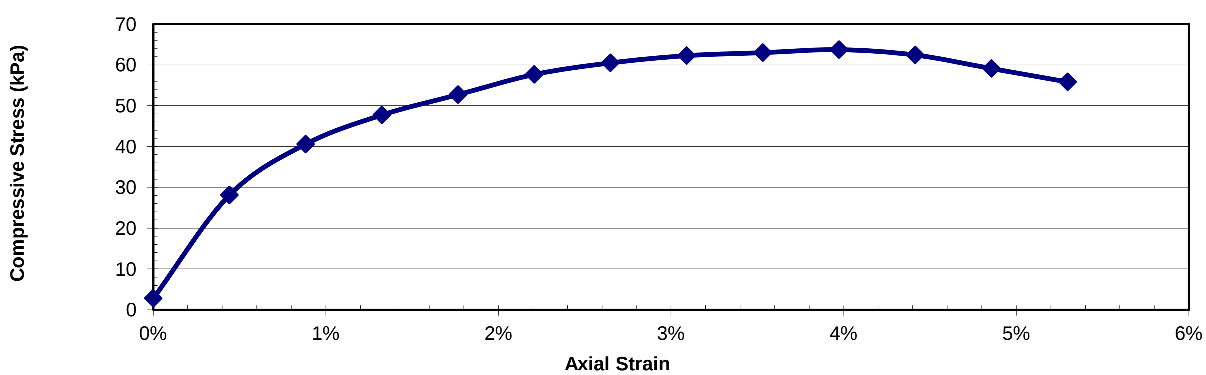
Stress	=	load/(corr. area)
Corr. Area	=	$A_o/(1 - \text{unit strain})$
Unit Strain	=	$\Delta L/L_o$
L_o/D_o	=	2.00
Strain Rate	=	0.88 %/min

Consistency	q_u, kPa
Very soft	0-24
Soft	24-48
Medium	48-96
Stiff	96-192
Very stiff	192-383
Hard	>383

Unconfined Compressive Strength, q_u = 64 kPa

Elapsed Time,min	Load-cell Dial Reading	Axial Load, kg	Strain Dial	Total Strain, mm	Unit Strain	Corrected Area, cm^2	Stress, kPa
0.0	6	1.18	0	0.00	0.00%	40.84	2.8
0.5	30	11.77	25	0.64	0.44%	41.02	28.1
1.0	42	17.06	50	1.27	0.88%	41.20	40.6
1.5	49	20.15	75	1.91	1.32%	41.39	47.7
2.0	54	22.36	100	2.54	1.77%	41.57	52.7
2.5	59	24.56	125	3.18	2.21%	41.76	57.7
3.0	62	25.89	150	3.82	2.65%	41.95	60.5
3.5	64	26.77	175	4.45	3.09%	42.14	62.3
4.0	65	27.21	200	5.09	3.53%	42.33	63.0
4.5	66	27.65	225	5.72	3.97%	42.53	63.8
5.0	65	27.21	250	6.36	4.41%	42.73	62.5
5.5	62	25.89	275	7.00	4.86%	42.92	59.1
6.0	59	24.56	300	7.63	5.30%	43.12	55.9

Post Test



Ver 12 - 2016/05/20

UNCONFINED COMPRESSION TEST REPORT

(Test Reference: ASTM D2166)

SNC·LAVALIN

Client:	City of Winnipeg
Project:	Community Row Pumping Station
Project #:	644548
Technician:	CH — Checked by: DH
Date:	May 1, 2017

Sample: TH01-09 at 6.1-6.7m

Specimen Data

Tare No:	=	ZV5
Weight of Tare, g	=	48.97
Weight of Specimen Wet + Tare, g	=	191.66
Weight of Specimen Dry + Tare, g	=	151.90
Water Content, %	=	38.6%

Average Pocket Pen Result	=	1.1	
Mass of Test Specimen, g	=	1061.76	
Wet Density, kg/m^3	=	1807	
Dry Density, kg/m^3	=	1303	
Specific Gravity	=	2.70	(assumed)
Degree of Saturation	=	0.97	

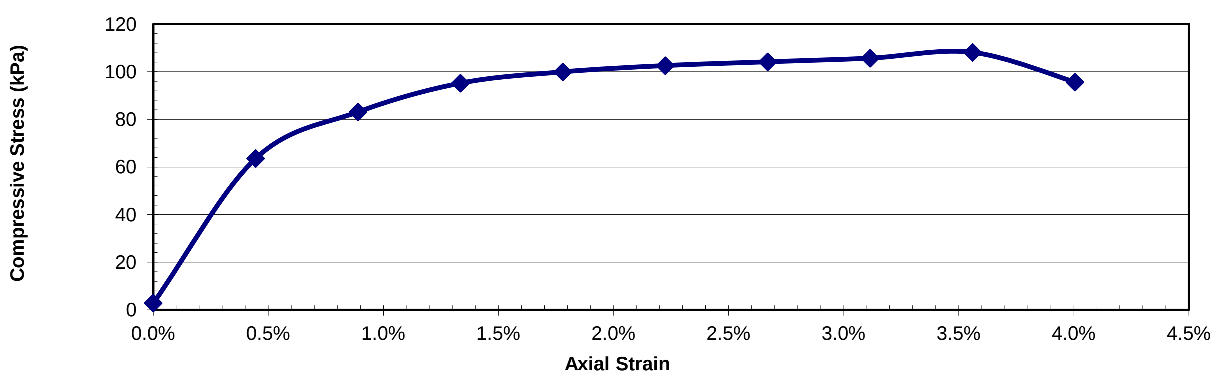
Initial Diameter, D_o, cm	=	7.24
Initial Area, A_o, cm^2	=	41.12
Initial Height, L_o, cm	=	14.29
Initial Volume, V_o, cm^3	=	587.74

Stress	=	load/(corr. area)
Corr. Area	=	A_o/(1 - unit strain)
Unit Strain	=	$\Delta L/L_o$
L_o/D_o	=	1.98
Strain Rate	=	0.89 %/min

Consistency	q_u, kPa
Very soft	0-24
Soft	24-48
Medium	48-96
Stiff	96-192
Very stiff	192-383
Hard	>383

Unconfined Compressive Strength, q_u = 108 kPa

Elapsed Time,min	Load-cell Dial Reading	Axial Load, kg	Strain Dial	Total Strain, mm	Unit Strain	Corrected Area, cm^2	Stress, kPa
0.0	8	1.18	0	0.00	0.00%	41.12	2.8
0.5	66	26.77	25	0.64	0.44%	41.31	63.6
1.0	85	35.15	50	1.27	0.89%	41.49	83.1
1.5	97	40.44	75	1.91	1.33%	41.68	95.2
2.0	102	42.65	100	2.54	1.78%	41.87	99.9
2.5	105	43.97	125	3.18	2.22%	42.06	102.5
3.0	107	44.85	150	3.82	2.67%	42.25	104.1
3.5	109	45.72	175	4.45	3.11%	42.45	105.6
4.0	112	47.00	200	5.09	3.56%	42.64	108.1
4.5	100	41.77	225	5.72	4.00%	42.84	95.6



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