APPENDIX

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GEOTECHNICAL REPORT

MANITOBA WATER SERVICES BOARD

BOISSEVAIN - MORTON WASTEWATER TREATMENT LAGOON

GEOTECHNICAL REPORT

CONFIDENTIAL

September 18, 2024

BOISSEVAIN - MORTON WASTEWATER TREATMENT LAGOON

GEOTECHNICAL REPORT

MANITOBA WATER SERVICES BOARD

TYPE OF DOCUMENT (VERSION)
CONFIDENTIAL

PROJECT NO.: CA0017817.1877
DATE: SEPTEMBER 18, 2024

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September 18, 2024

CONFIDENTIAL

Manitoba Water Services Board
The Manitoba Water Services Board
Unit 1A-2010 Currie Blvd.
Brandon, Manitoba R7B 4E7

Attention: Nathan Wittmeier, M.T.S., P.Eng.

Dear Sir:

Subject: Geotechnical Report for Boissevain - Morton Wastewater Treatment Lagoon

WSP is pleased to submit this Geotechnical Report for Boissevain - Morton Wastewater Treatment Lagoon to be located Boissevain, Manitoba. If you have any questions, please contact the undersigned directly at 431-688-5172.

Yours sincerely,

[redacted]

Linh Trinh, M. Eng., P. Eng.
Senior Geotechnical Engineer

LT

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REVISION HISTORY

FIRST ISSUE

2024-09-18	Issue to Client			
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2024-09-18

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1 INTRODUCTION

As authorized by Nathan Wittmeier, M.T.S., P.Eng. of Manitoba Water Services Board, WSP Canada Inc. (WSP), conducted a geotechnical investigation for the Boissevain - Morton Wastewater Treatment Lagoon.

The scope of work for the geotechnical investigation was defined initially in WSP's proposal WPG2024.014 dated 15 March 2024 including a new lagoon site, wastewater lift station, and proposed area for the existing lagoon expansion (to the south of the existing lagoon). The scope of work was finally determined with cancelling the new lagoon site and adding 2 borrow sites.

The purpose of the investigation was to determine the subsurface soil and groundwater conditions at the Site in order to provide geotechnical recommendations for design and construction of a new lift station, upgrading the existing lagoon, and evaluation of the quality of proposed clay borrow sources for the lagoon construction.

2 PROJECT DESCRIPTION

2.1 SITE DESCRIPTION

It is understood that the wastewater treatment lagoon (WWTL) is proposed to be located to the southwest of the community. At the time of the investigation, the site consisted of an existing lagoon, an existing lift station to the north and undeveloped land to the south of existing lagoon. The property is bordered by farmland within Aberdeen St on the north, McKay St. on the east, Rd 15 N on the south, and Rd 116 W to the west. Topographic relief across the site was visually estimated to be approximately 1.0 m, not including ditches; and overland drainage was visually inferred to be generally southwards towards Rd 15 N.

It is also understood that two clay borrow sources, NW13-3-20W and NW14-3-20W, are proposed for lagoon construction. The source NW13-3-20W is 800 m approx. to the southeast of the WWTL site, and NW14-3-20W is 800 m approx. to the southwest of the WWTL site.

2.2 PROJECT DESCRIPTION

The layout of the proposed development is shown in Appendix A. Based on preliminary information provided to WSP, it is understood that the project will consist of a new lift station proposed to the north of the existing lagoon (in vicinity of the existing lift station) and upgrades of the existing lagoon. No design of the new lift station is available at the time of writing this report. Based on the drawings provided for the existing lift station, the existing lift station was constructed as a concrete shaft with diameter of 4.2 m approx. and embedded 6.0 m approx. below ground surface. Based on preliminary design drawings, the lagoon upgrades involve relining the existing secondary cell, raising the berms and decommissioning the existing primary cell. The lagoon facility will be expanded to the south with SAGR cells, a operations building, and potentially a new aerated cell. The availability of the suitable clay material for construction of a compacted lagoon liner will impact to the selection of upgrade options.

3 GEOTECHNICAL INVESTIGATION

Prior to initiating drilling, WSP notified public utility providers (i.e. Manitoba Hydro, Westman Media Cooperative Ltd (MB) of the intent to drill in order to clear public utilities. A private utility locator was also retained by WSP to scan the proposed test hole locations for the presence of detectable private utilities. All field activities were completed without contact with underground utilities.

From 6 May 2024 to 8 May 2024, WSP supervised the drilling of eight (8) test holes for the lift station and existing lagoon sites at the approximate locations illustrated in Appendix A. Test holes TH24-01 and TH24-02 was advanced to 12.6 m below existing grade (a companion test hole, TH24-01A, was advanced to 7.6 m below existing grade immediately adjacent to TH24-01 to permit installation of a standpipe piezometer in the vicinity of the lift station), while test holes TH24-03 to TH24-08 were advanced to approximately 6.1 m below existing grade in the vicinity of the proposed lagoon upgrades.

Between 1 and 2 August 2024, WSP also supervised the drilling of seven (7) test holes for each of two borrow sites, NW13-3-20W and NW14-3-20W, fourteen (14) test holes in total, at the approximate locations illustrated on in Appendix A. Test hole TH24-14 and TH24-17 were initially proposed for the site NW14-3-20W but were removed from the scope by the designed team. All borrow test holes were advanced to 4.6m below existing grade.

The test holes were drilled using track mounted drill rigs (Acker MP5, Mobile B57, and Mobile B48) equipped with 125 mm solid stem augers and 200 mm hollow stem augers, operated by Paddock Drilling Ltd. of Brandon, Manitoba. During drilling, WSP field personnel visually classified the soil stratigraphy within the test holes in accordance with the Modified Unified Soil Classification System (MUSCS); and recorded observed seepage and sloughing conditions. Soil sampling consisted of grab samples of the auger cuttings in all test holes, split spoon samples obtained in combination with Standard Penetration Tests (SPTs) in cohesionless or till soils (except borrow test holes) and relatively undisturbed Shelby Tube samples of the clay and clay till obtained at select depths in lift station, lagoon upgrade and borrow test holes. All samples were retained in sealed plastic bags and shipped to WSP's Oak Bluff laboratory for review and selected testing.

The in-situ relative consistency of cohesive soil was evaluated during drilling using a pocket penetrometer. Standard Penetration Tests (SPT's) were conducted in cohesionless soils to characterize the density of till deposits (except borrow test holes). For each SPT, a standard 63.5 kg hammer was repeatedly dropped 760 mm to drive a standard 50.8 mm diameter thick-walled sampling tube through the soil over up to three consecutive depth intervals of 150 mm each. The number of drops, or blows, required for the hammer to drive the sampler through each 150 mm interval was recorded. The recorded SPT results are shown on the test hole logs as the SPT 'N' value, which represents the number of blows to drive the sampler through the final two 150-mm intervals or partial increments in cases where 50 blows produced less than 150 mm penetration.

Two standpipe piezometers were installed for lift station site, in which one was placed within the top sand layer in test hole TH24-02 to a depth of 3 m and another within the clay till at a depth of 7.3 m to intersect a water bearing near sand layer near the bottom of test hole TH24-01A. One standpipe piezometer was also installed within a sand layer at 4.5 m in test hole TH24-03 for lagoon site. A return trip to the site to monitor the groundwater level (GWL) was completed on August 28, 2024, the results of which are summarized in Table 4-4.

Upon completion of drilling, the depths of seepage and sloughing zones as well as depths to accumulated slough and groundwater level were measured prior to backfilling. The piezo-installed test holes were backfilled with bentonite to the target monitoring depth, then by silica sand within 0.3m below the bottom and above the top of screened Casagrande tip, followed by bentonite to the ground surface. Other test holes were backfilled to grade with auger cuttings and a bentonite seal at surface.

Following completion of the field drilling program, a laboratory testing program was conducted. The laboratory testing program within the area of the lift station and lagoon upgrade (TH24-01 to TH24-08) consisted of:

- moisture content determinations on all grab samples,
- eight Atterberg Limits,

- five Hydrometer tests, and
- four sieve analysis on select samples from test holes.

For borrow site NW14-3-20W lab testing consisted of:

- five Moisture Contents,
- two Atterberg Limits,
- two Hydrometers,
- two Standard Proctors, and
- two hydraulic conductivity tests. The conductivity tests were completed on remolded samples compacted to 95% of Standard Proctor Maximum Dry Density (SPMDD).

No tests were required for the borrow site NW13-3-20W.

The test hole logs presented in Appendix B include the sampling, field testing, laboratory test results, and subsurface conditions encountered at the test hole location. Summaries of the terms and symbols used on the test hole logs and of the Modified Unified Soil Classification System are also presented in Appendix B. The laboratory test reports are provided in Appendix C.

4 SUBSURFACE CONDITIONS

4.1 STRATIGRAPHY

Consistent with the regional soil deposits, the stratigraphy observed at the test holes consisted of the following, in descending order from grade level:

- Sand Fill
- Organic Clay
- Clay
- Silt
- Sand
- Clay Till

A brief description of each of the soil layers listed above is presented in the following sub-sections separately for the lift station, existing lagoon, and borrow sources. For detailed descriptions, the test hole logs in Appendix B should be consulted.

4.1.1 LIFT STATION SITE

Sand Fill

Sand fill was present at the ground surface in test hole TH24-01 and was 300mm thick.

Organic Clay

Organic clay was encountered at the ground surface in test holes TH24-02 and was 1.1 m thick. The organic clay was silty, contained trace sand, medium to high plastic, moist, stiff, dark brown, and contained occasional grass roots and rootlets. Moisture contents in the organic clay ranged from 16% to 18%.

Clay

Clay was encountered below the sand fill in test hole TH24-01extending to a depth of 1.2m. The clay was silty with trace sand, medium plastic, moist, very stiff, and greyish grey. The moisture content for the clay was of 15%.

Silt

Silt was encountered below the clay layer in test holes TH24-01 extending to a depth of 2.0m. The silt contained trace to some clay, and trace sand, and was low plastic, stiff, and dark grey. The moisture content for the silt was of 18.7%.

SAND

Sand was encountered below the organic clay in test hole TH24-02, extending to a depth of 3.0 m. The sand contained trace silt, trace clay, and was poorly graded, fine grain, moist to wet, inferred as loose, and dark brown. The moisture content for the sand ranged from 15% to 19%

CLAY Till

Clay till was present below the silt in test hole TH24-01 and below the sand in test hole TH-02, extending to the termination depth of 12.6m in each case. The clay till was described as silty, generally contained trace sand and trace gravel, medium to high plastic, moist, firm to hard, and brown to grey. Standard penetration test (SPT) ‘N’ values ranged from 6 to 49 and was generally greater than 40 below a depth of about 6 m. Moisture contents in the clay till ranged from 10% to 25%.

4.1.2 EXISTING LAGOON SITE

Organic Clay

Organic clay was encountered at the ground surface in test holes TH24-05 and TH24-06 and was 0.2 m to 0.6m thick. The organic clay was silty, medium plastic, moist, stiff, dark brown, and contained occasional grass roots and rootlets. Moisture contents in the organic clay was about 50%.

Clay

Clay was encountered from surface in test hole TH24-03, and below the organic clay in test holes TH24-05 and TH24-06 extending to depths ranging from 1.2m to 2.1m. The clay contained silt amounts ranging from silty to and silt, had trace sand and was medium plastic, moist, firm to stiff, and dark brown to grey. Clay in test hole TH24-03 contained occasional roots. The moisture content for the clay ranged from 25% to 39%.

Silt

Silt was encountered at the surface in test holes TH24-04, TH24-07, TH24-08, below the clay layer in test holes TH24-03, TH24-05, and again below the sand layer in test hole TH24-03, extending to depths ranging from 1.5m to 5.5m. The silt contained clay amounts ranging from clayey to trace clay, had trace sand to some sand, and was low plastic, soft to stiff, and brown to dark grey. The moisture content for the silt ranged from 11% to 40% depending on clay content.

SAND

Sand was encountered below the clay layer in test hole TH24-06 and below the silt layer in test holes TH24-03 to TH24-05, TH24-07 and TH24-08 extending to depths ranging from 3.7 m to 6.4 m. The sand contained gravel amounts ranging from gravelly to no gravel, and had some silt to no silt, and trace to no clay, and was poorly graded, fine to coarse grain, very moist to wet, loose to compact, and brown to grey. Standard penetration test (SPT) ‘N’ values ranged from 4 to 24. The moisture content for the sand ranged from 14% to 23%.

CLAY Till

Clay till was present below the sand layer in test holes TH24-04 to TH24-08 and below the silt layer in test hole TH24-03 extending to depths explored ranging from 6.1m to 6.7m. The clay till was described as silty, generally contained trace sand and trace gravel, and was low plastic, moist, firm to very stiff, and dark grey. Standard penetration test (SPT) was conducted in test holes TH24-03 to TH24-07 where the SPT ‘N’ values ranged from 7 to 26. Moisture contents in the clay till layer ranged from 8% to 19%.

4.1.3 BORROW SOURCE NW14-3-20W

Sand Fill

Sand fill was present at the ground surface in test hole TH24-10 and was 1.2m thick. The sand fill contained some gravel, and was fine grained, inferred as loose, and brown.

Organic Clay

Organic clay was encountered at the ground surface in test holes TH24-09, TH24-12, TH24-13, and TH24-15 and was 0.2 m to 0.4m thick. The organic clay contained silt amounts ranging from silty to and silt, trace sand, and trace gravel, and was medium plastic, damp moist, firm to very stiff, dark brown, and contained occasional grass roots and rootlets.

Organic Silt

Organic silt was encountered at the ground surface in test holes TH24-16 and was 0.5 m thick. The organic silt contained some clay, and was medium plastic, moist, firm, and dark brown.

Clay

Clay was encountered from surface in test hole TH24-11, and below the sand fill in test hole TH24-10, below the organic clay in test holes TH24-09, TH24-12, TH24-13, and TH24-15 extending to depths ranging from 1.5m to 4.6m (at the termination depths). The clay contained silt amounts ranging from silty to and silt, and trace sand. Moisture content of the clay ranged from 16% to 29% increasing with depth.

Per the investigation, eight of nine Atterberg Limits tests for the clay within this site, including the lift station, the lagoon extension, and borrows, yielded the liquid limit of 30 with the exception of one sample (TH24-12, sample 5@2.1m) where the liquid limit was greater then 50. This implies that the high plastic clay sample (TH24-12, sample 5@2.1m) might not represent for the clay material in this site and that low to medium plastic clay material is dominant.

Atterberg Limits, hydrometer grainsize, standard proctor and hydraulic conductivity were conducted for two samples. The hydraulic conductivity was performed on remolded specimens compacted to 95% Standard Proctor Maximum Dry Density. Summary of the test results are provided in Table 4-1 and Table 4-2.

Table 4-1: Summary of Atterberg and Hydrometer Tests

Sample ID	Atterberg Tests			Hydrometer Tests		
	LL (%)	PL (%)	PI(%)	Sand (%)	Silt (%)	Clay (%)
TH24-09/3m	36	15	21	27	42	31
TH24-12/2.1m	70	24	46	1	52	47

Table 4-2: Summary of Proctor and Hydraulic Conductivity Tests

Sample ID	Proctor Test		Hydraulic Conductivity Test @ 95% of SPMDD
	Maximum Dry Density (kg/m^3)	Optimum Moisture Content (%)	Hydraulic Conductivity (x10^{-7} cm/sec)
TH24-09/Combined clay samples	1733	15.9	0.592
TH24-10 and TH24-12/Combined clay samples	1633	18.6	0.476

It is noted that the moisture contents (MC) of the clay are on the wet side of the optimum moisture contents (OMC). Accordingly, the clay will require moisture conditioning to bring the MC to approximately the OMC for the compaction.

The conductivity values are considered represented for the low plastic clay that are likely dominant within this site as discussed above.

Silt

Silt was encountered below a sand layer in test holes TH24-09, below the clay layer in test holes TH24-13 and TH24-15, and below the organic silt layer in test hole TH24-16, extending to depths ranging from 2.6m to 4.6m (termination depth). The silt contained clay amounts ranging from clayey to some clay, and was low to medium plastic, soft to stiff, and brown to light grey.

SAND

Sand was encountered below the clay layer in test hole TH24-09 and TH24-11, and below the silt layer in test hole TH24-16, extending to depths ranging from 3.3 m to 3.8 m. The sand contained trace gravel, and trace silt, and was poorly graded, fine grained, very moist to wet, inferred as loose, and brown.

CLAY Till

Clay till was present below the sand layer in test holes TH24-11 and TH24-16, below the silt layer in test holes TH24-13 and TH24-15, extending to the termination depth of 4.6 m. The clay till was described as silty, generally contained trace sand and trace gravel, and was medium plastic, moist, very stiff to hard, and brown to grey.

4.1.4 BORROW SOURCE NW13-3-20W

Organic Clay

Organic clay was encountered at the ground surface in test hole TH24-19 and was 0.4 m thick. The organic clay was silty, with trace sand, medium plastic, moist, stiff, dark brown, and contained occasional grass roots and rootlets.

Organic Silt

Organic silt was encountered at the ground surface in test hole TH24-22 and was 0.4 m thick. The organic silt contained trace clay, and trace sand, and was low plastic, moist, soft, dark brown, and contained occasional roots.

Clay

Clay was encountered from surface in test hole TH24-21, below sand layers in test holes TH24-20, TH24-23, and below a silt layer in test hole TH24-22 extending to depths ranging from 1.5m to 4.6m (at the termination depths). The clay contained silt amounts ranging from silty to and silt, trace sand, and trace gravel, and was medium plastic, moist, firm to very stiff, and brown to grey.

Silt

Silt was encountered below the organic clay in test hole TH24-19 and below the organic silt layer in test hole TH24-22, extending to depths ranging from 1.4m to 1.5m. The silt contained clay amounts ranging from trace clay to some clay, and contained sand amount from some sand to sandy, and was low plastic, soft to stiff, and grey.

SAND

Sand was encountered from ground surface in test holes TH24-18, TH24-20, TH24-23 and TH24-24, and within the clay layer at TH24-20, extending to depths ranging from 2.4 m to 4.6 m (the termination depth). The sand contained silt and clay amounts ranging from trace clay to some clay, and trace silt to silty, and was poorly graded, fine grained, damp to moist, inferred as loose to compact, and brown to dark brown.

CLAY Till

Clay till was present below the silt layer in test hole TH24-19 and below the clay layer in test hole TH24-22, extending to the termination depth of 4.6 m. The clay till was described as silty, generally contained trace sand and trace gravel, and was medium plastic, moist, stiff to very stiff, and brown to grey.

4.2 TEST HOLE TERMINATION

Lift station test holes (TH24-01 and TH24-02) terminated at a depth of 12.6 m below existing grade in clay till. Test holes for the existing lagoon (TH24-03 to TH24-08) terminated at depths of 6.1 m to 6.7 m below existing grade in clay till. In the borrow site NW14-3-20W, test holes (TH24-09 to TH24-16), except test holes TH24-14 and TH24-17 that were cancelled by the designed team) terminated at a depth of 4.6 m below existing grade in clay, or silt, or clay till. Test holes for borrow site NW13-3-20W (TH24-18 to TH24-24) terminated at a depth of 4.6 m below existing grade in clay, or clay till, or sand. No auger refusal was encountered in all the test holes.

4.3 GROUNDWATER AND SLOUGHING CONDITIONS

Seepage and sloughing conditions were noted during drilling, and the depths to the accumulated slough and water levels about ten minutes after completion were measured prior to backfilling. Recorded observations are summarized in **Error! Reference source not found.**.

Table 4-3: Sloughing and Short-term Groundwater Observations

Test Hole ID	Drill Depth (m)	During Drilling		Upon Completion	
		Sloughing Zone (m)	Seepage Zone (m)	Depth Open (m)	Depth to Groundwater (m)
List Station					
TH24-01	12.6	1.2 – 2.0	6.3 – 6.7	11.3	8.8
TH24-02	12.6	1.1 – 3.0	1.1 – 3.0	8.2	1.5
Existing Lagoon					
TH24-03	6.6	2.3 – 4.6	2.3 – 4.6	3.0	0.3
TH24-04	6.7	1.5 – 4.7	1.5 – 4.7	1.5	0.6
TH24-05	6.6	2.1 – 4.0	4.0 – 6.4	3.7	0.6
TH24-06	6.6	1.2 – 3.7	1.2 – 3.7	3.0	0.3
TH24-07	6.1	2.0 – 4.4	2.0 – 4.4	1.5	0.3
TH24-08	6.1	1.2 – 4.6	1.2 – 4.6	2.4	1.5
Borrow site 14-3-20W					
TH24-09	4.6	3.5 – 3.7	3.5 – 3.7	3.7	2.4
TH24-10	4.6	2.4 – 2.6	2.4 – 2.6	4.4	3.8
TH24-11	4.6	3.2 – 3.7	3.2 – 3.7	1.5	1.2
TH24-12	4.6	1.5(1)	No	3.4	No Accumulation(2)
TH24-13	4.6	1.5 – 3.0	1.5 – 3.0	2.1	No Accumulation(2)
TH24-15	4.6	2.3 – 3.4	2.3 – 3.4	4.3	No Accumulation(2)

Test Hole ID	Drill Depth (m)	During Drilling		Upon Completion	
		Sloughing Zone (m)	Seepage Zone (m)	Depth Open (m)	Depth to Groundwater (m)
TH24-16	4.6	2.6 – 3.4	2.6 – 3.4	4.0	1.8
Borrow site 13-3-20W					
TH24-18	4.6	None Observed	None Observed	4.6	No Accumulation
TH24-19	4.6	None Observed	None Observed	4.6	No Accumulation
TH24-20	4.6	None Observed	None Observed	4.6	No Accumulation
TH24-21	4.6	3.0 – 3.4	3.0 – 3.4	4.3	4.1
TH24-22	4.6	0.5 – 1.5	0.5 – 1.5	1.2	1.1
TH24-23	4.6	2.4(1)	None Observed	0.2	No Accumulation(2)
TH24-24	4.6	4.4(1)	None Observed	4.4	No Accumulation(2)

Note: (1) Caving

(2) No water accumulation above observed open test hole depth prior to backfilling

The groundwater levels monitored in the standpipe piezometers on August 28, 2024, are summarized in Table 4-4.

Table 4-4: Groundwater Monitoring Data

Piezo Installed in Test Hole	Screening Depth (m)	Groundwater Level (m) Below Grade	Soil Unit
List Station			
TH24-01a	5.8 – 7.3	2.53	Sand Seam/Clay Till
TH24-02	1.2 – 3.0	Dry	Sand
Existing Lagoon			
TH24-03	4.7 – 5.0	1.68	Sand

It should be noted that groundwater levels can fluctuate annually, seasonally, or as a result of construction activity.

5 DESIGN AND CONSTRUCTION RECOMMENDATIONS FOR THE LIFT STATION

5.1 GENERAL DISCUSSION

Soil conditions encountered at the lift station test holes (TH24-01 and TH24-02) consisted of layers of clay, silt, sand extending to depths ranging from 2.0 m to 3.0 m below ground surface, underlain by medium to high plastic clay till. Sand lensing was noted (TH24-02) within the clay till from 6.3 m to 6.7 m from ground surface. Seepage and sloughing were recorded during drilling within shallow silt and sand layers as well as in the sand seams within clay till.

As mentioned above, no proposed designs are available at the time of writing this report. Therefore, based on the existing lift station design (see Section 2.2), the following recommendations are based on an assumption that the lift station will be a shaft with a minimum diameter of 4.2 m supported by a raft foundation with a minimum embedment depth of 6.0 m. In this regard, and expected light foundation loads, it is expected that the lift station foundation could be designed on the basis of maintaining a net bearing pressure near zero (i.e. weight of structure is approximately the same as the weight of the soil being removed).

For excavation, given the site conditions and associated risks and challenges outlined above, the depth of the lift station, and the fact that the construction may last months, shoring alternatives are considered preferable. Technically, sloped excavation might be considered. However, this option will generate a huge excavation and backfill works compared to shoring. Furthermore, since excavation should maintain open for months for construction, a comprehensive monitoring program, dewatering plan, etc. shall be prepared and implemented throughout the construction period until the backfill is completed. Nevertheless, sloped excavation might pose instability risks for the primary cell of the existing lagoon that located less than 20 m west of the proposed list station. For those reason, sloped excavation is not recommended in this report.

Shoring alternatives including permeable wall (e.g. soldier piles with timber slagging), relatively impermeable wall (e.g. steel sheet pile), and impermeable wall (e.g. Circular Steel Sleeve) can be considered for this site. The excavations extended to a depth of about 6.0 m below grade or deeper will be subject to groundwater issues given that the elevation of the groundwater table was determined to be about 1.68 m below existing grade in the sand layer (referred to TH24-03) and 2.53 m in sand seams (TH24-01) as monitored on 28 August 2024. Shallow as 0.3m below ground surface of groundwater levels should also be expected based on the observation during the drilling at the south site of the existing lagoon. Unless the groundwater level will be controlled to or below the excavation base, the groundwater can easily penetrate through the gaps between timbers and might cause piping for the soil beyond the wall. Furthermore, since the timbers will not be extended below the excavation base, the soldier piles with timber lagging are impossible to control boiling that is considered possible below the excavation base. It is important to note that, if it is not to be controlled properly or eliminated, potential boiling/piping can pose instability risks to not only the supporting wall but also the existing primary cell. For those reason, soldier piles with timber lagging is provided in this report as a potential option subjected strictly to applicable conditions outlined herein.

Sheet piled wall, particularly steel sleeve, is more effective to prevent the seepage from infiltrating into the excavation. Those methods can entirely extend and embed below the excavation base, and therefore, will be more effective to control and/or eliminate the instability risks that might result associated with the seepage. Therefore, sheet piles or steel sleeve are considered suitable for this site. However, it is important to note that even though the sheet piles are preferrable over the soldier piles option for this site, they carry challenges such as encountering cobbles/boulders or hard driving and the contractor should prepare for those difficulties of the installation. Also, seepage might be anticipated minimal through the wall where sheet piles applied, the seepage at the bottom during the excavation is anticipated still significant given the groundwater condition encountered. Therefore, dewatering is still required for sheet piles or steel sleeve is used.

The following sections provide discussion and recommendations for design and construction of the proposed lift station.

5.2 EXCAVATION STABILITY

Given the soil condition, sloughing and seepage conditions, and groundwater encountered, stability against shear failure, boiling/piping, and base heave should be checked. Those instability and excavation challenges will be affected by the following factors:

1. Permeable water bearing shallow silt and sand layers may lead to caving, sloughing, and seepage as construction proceeds.
2. Permeable sand seams (at 6.4m approx. depth) within clay till may lead to significant seepage, softening and piping and/or heave at the excavation base that may further lead to difficulty in establishing a suitable bearing surface for foundation construction. In additional, the water level in the sand seam is about 2.5 m below grade and might cause buoyant conditions particularly when the lift station is empty. Therefore, the lift station design will need to consider buoyancy.
3. Very stiff to hard clay till is expected to be present at or near the expected depth of the raft foundation and may impact to selection of construction methods.
4. Although boulders and cobbles were not encountered during drilling they might be present within the clay till and might pose challenges for the construction.
5. These risks will be further influenced by the construction methods selected, timing and sequencing and will depend on time of year, as water levels may vary significantly on a seasonal basis.

Generally, there are three base failure modes that must be evaluated specific to the design of a supported excavation as follows:

1. Shear failure,
2. Piping, and
3. Heave.

In addition to the above failure modes, general seepage from the excavation sides and base will need to be controlled in order to maintain undisturbed bearing conditions throughout construction of the lift station.

The following sections discuss each of the above noted design considerations and provide comments regarding suitable shoring systems to protect against each as applicable.

5.2.1 BASE STABILITY AGAINST SHEAR FAILURE

The stability of the excavation base against shear failure must be evaluated to confirm a safe excavation base condition. This failure mechanism occurs as a result of inadequate resistance of the loads imposed by the differences in grades between the inside and outside of the excavation. According to Canadian Foundation Engineer Manual, Edition 5 (CFEM), if the FS against base shear failure is less than 1.5, then the depth of penetration of the support system MUST extend below the base of the excavation. In designing a shoring system for stability against base shear failure, a FS greater than 2.0 should generally be targeted. If FS is less than 2.0, substantial deformation may occur.

For this Site, the base of the excavation would be founded within very stiff to hard clay till and as such, the FS against base shear failure is greater than 2.0. Therefore, the design requirement for this mode of failure is met without extending the shoring below the intended base of the excavation. In this case, either a soldier pile and timber lagging or driven sheet pile shoring system may be possible.

5.2.2 BASE STABILITY AGAINST PIPING FAILURE

Given the groundwater encountered as noted, boiling is potential risk for the excavation base. The stability against boiling can be evaluated by different methods and different recommended factor of safety Fs=1.3 or higher is normally recommended for Terzaghi method and 2.0 or higher for critical hydraulic gradient method. A preliminary calculation based on critical hydraulic gradient method with a target Fs=2.0 for the groundwater level of 1.68m below ground surface indicated that the required embedment of supporting wall will be 2.0m to 3.4m approx. below excavation base when the excavation depth ranges from 6.0 to 9.0m (Figure 5-1). In this case, a sheet pile shoring system or steel sleeve is considered suitable.

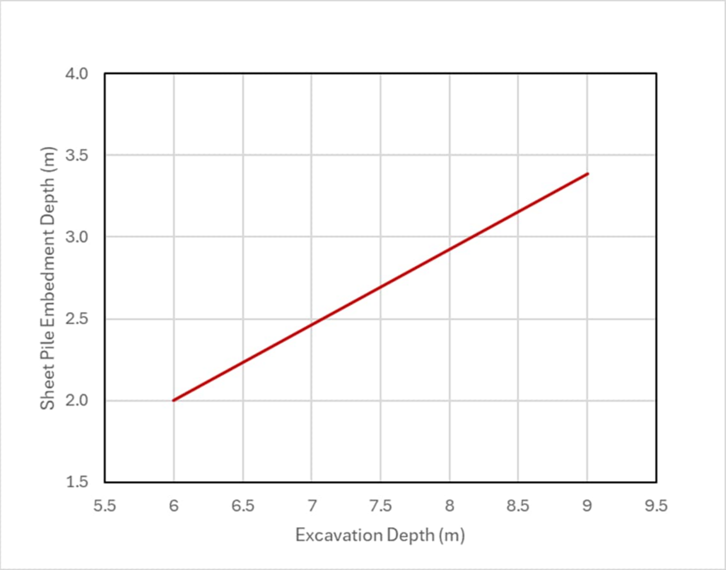

Figure 5-1 Wall Embedment Depth vs Excavation Depth

It is important to note that the Figure 5-1 is presented for the specific conditions as noted above. This assessment should be revised in the event that the lift station configuration is revised in the next stage of design. In this regard, the groundwater level should be updated for verification prior to and during the construction to verify if the calculation output is still valid. Furthermore, given the preliminary nature of the above assessment, WSP recommends that a seepage analysis be completed during the final design phase to verify the embedment depth against piping once the detailed design configuration of the lift station is available.

5.2.3 SOILS HEAVE AT THE EXCAVATION BASE

Soil heave can occur at the base of an excavation in cases where an impervious layer (i.e. clay) is present at the excavation base, overlying a pervious layer (i.e. sand or till) which contains groundwater under pressure. For this case, given the anticipated sand seam containing groundwater encountered to the depth of 6.7m, if bearing depth to be approximately 7m or deeper, the bearing soil is anticipated to be clay till therefore soil heave due to this mechanism is not anticipated to be an issue.

In case the bearing depth is about 6.0m, the remaining clay till on top of the sand seam might be in order of 0.3 m thick. In this scenario, the upward water pressure at the level of 6.3 m below grade (top of the sand seam) will be in order of [(6.3-2.53) x 9.81 = 37 kPa] while the soil pressure at this level will be 0.3 x 20 = 6 kPa approximately. That means base heave failure due to groundwater in sand lenses would be a potential risk if the bearing depth is about 6.0 m or shallower. Therefore, a minimum 7.0 m bearing depth is recommended to avoid the sand lenses is recommended for this site to protect the excavation again the potential heave failure.

5.2.4 SEEPAGE

As outlined in Section 5.1, significant seepage should be expected through the wall unless sheet piles or steel sleeve are used; significant seepage should also be anticipated the through the excavation base. The seepage rate will depend on the actual soil conditions at the excavation, most importantly those at the base. Depending on the construction period, significantly higher water levels could also be encountered (i.e. during spring or summer flood events). Groundwater from sand seams within the clay till needs to be controlled so that the excavation can be safely constructed without disturbance to the excavation base. It is important to note that, the seepage should be expected and should be control through the excavation till the lift station construction is completed.

5.2.5 SUMMARY OF THE SHORING SYSTEM

According to the aforementioned discussions, an appropriate shoring system for the lift station can be summarized as follows:

1. The proposed lift station will be bearing directly on the clay till at a depth of 7.0m or deeper; and
2. If groundwater level can be controlled below the base of the excavation and then a braced soldier pile with timber lagging system might be applied, otherwise a braced sheet pile system, or circular steel sleeve should be considered.

5.2.6 LATERAL EARTH PRESSURE FOR TEMPORARY SHORING

The distribution of lateral earth pressure on a shoring system depends on many factors including, but not limited to, the soil type, groundwater conditions over the depth of the shoring, surcharge loading at the surface, rigidity of the system, and the target degree of shoring wall movement resulting in full, or partial, development of active earth pressures.

Based on the premise that the shoring will consist of steel sheet piles or soldier piles with timber lagging systems that are braced internally with a system of steel walers and/or struts in order to restrain shoring movements, or a circular steel sleeve, the ‘apparent’ distribution of earth pressure to be resisted by a braced shoring system given layered soils should be calculated according to Section 20.8.1.4, Retaining Structures with Internal Braces of the CFEM, Edition 5, utilizing the apparent earth pressure distributions shown in Figure 20.20d and the following soil parameters. The Figure 20.20d in the CFEM is reproduced in Appendix D of the report.

Table 5-1: Lateral Earth Pressure Coefficients on the Lift Station Shoring

Soil Parameters	**Clay**	**Silt**	**Sand**	**Clay Till**
Total unit weight, γ (KN/m^3)	18	19	19	22
Submerged unit weight, γ’ (KN/m^3)	8.2	9.2	9.2	12.2
Internal friction angle, φ’	18	22	30	25
At rest earth pressure coefficient, Ko	0.69	0.63	0.5	0.58
Active earth pressure, Ka	0.53	0.45	0.33	0.41

Soil Parameters	Clay	Silt	Sand	Clay Till
Passive earth pressure, Kp (1)	1.26	1.47	2.0	1.64

Note: (1) the Kp has been reduced by a factor of 1.5 to account for strain compatibility.

The passive resistance is developed by that portion of the sheet or soldier pile below excavation base. In the case of soldier piles and lagging, the passive resistance should be taken to act on the diameter of the embedded portion of the soldier pile below the lowest excavation grade. A geotechnical resistance factor of 0.5 should be applied to the calculated passive resistance to determine the factored ultimate geotechnical resistance.

Total unit weights of the soils should be used above the water table. A combination of submerged soil unit weights and horizontal pressure resulting from the design static water level should be used below the water table. For purposes of temporary shoring design, the water level can be assumed to be 1.6 m below grade. However, the groundwater should be monitored over a period of time leading up to the design of the shoring system to determine if any change to the groundwater table has occurred. Prior to the temporary shoring construction, the groundwater conditions should be monitored to confirm the estimations/assumptions made during the design phase are still valid. If the water table rises to an elevation higher than those estimated in the design phase, the entire shoring system should be evaluated to confirm whether the design remains appropriate.

The apparent earth pressure distribution used in design will adopt a combined trapezoidal/triangular distribution. In general, the lateral earth pressures are calculated as follows:

$$\sigma_h = K \times \sigma_v' + h \times \gamma_w \qquad \text{Equation 1}$$

Where:

K : Earth Pressure Coefficient

σ'_v : Vertical Effective Stresses

h : height of design groundwater level above the base of the wall

γ’: Submerged unit weight (Table 5-1)

γ_w : Unit weight of water (9.81 kN/m^3)

The value of K used in the Equation 1 above will be influenced by the amount of lateral wall movement that is considered applicable.

1 If moderate wall movements (i.e. 1.0% to 2.0% of the excavation depth) can be permitted or are expected, the pressure may be computed using the coefficient of active earth pressure, K_a.

2 If services adjacent to the excavation exist at a shallow depth, at a distance less than H (height of the wall) behind the top of the wall, and not closer than 0.5 H and some movements (i.e. 0.3% to 1.2% of the excavation depth) of services can be tolerated, the lateral earth pressure may be calculated using a coefficient determined as follows:

$$K = 0.5 \times (K_a + K_o) \qquad \text{Equation 2}$$

3 If services exist at a shallow depth, or if there are adjacent existing foundations at a distance less than 0.5 H behind the top of the wall or if movements of services are intolerable, the pressure should be computed using the coefficient of earth pressure at rest, K_o.

For frozen soil, the “at rest” case consisting of Ko = 1.0 should be used to determine the lateral earth pressure on temporary shoring (see Section 5.3.4).

It is important to note that the lateral earth pressures should have the appropriate load factor applied as outlined in Section 5.2.8.

WSP can provide the lateral earth pressures distributions of the proposed shoring system once the details of the excavation and shoring type are finalized.

5.2.7 SURCHARGE LOADS

In addition to earth pressures, lateral stresses generated by any applicable surcharge loads also need to be evaluated in the design. The surcharge considered should include the effects of loads from construction equipment and any other loads that may be transferred to the walls of the excavation during the construction period.

For line or point surcharge loads, the lateral pressures should be determined using the relationships given in Appendix D. In the case of uniformly distributed surcharge loads, such as those acting on the surface of the retained soil, the induced lateral earth pressure may be determined by multiplying the surcharge load by the appropriate earth pressure coefficient.

5.2.8 LOAD FACTORS

For the Limit States Design procedure for walls, loads, load combinations, and load factors should be applied in accordance with applicable designed standards. For preliminary calculation, the following recommendations might be applied.

1. For earth loads acting on walls, a Load Factor of 1.25 is recommended for sustained loads.
2. For hydrostatic loads acting on walls, a Load Factor of 1.1 is recommended.
3. For live surcharge loads acting on walls, the Load Factor of 1.5 should be used.

The above load factors should be applied to loads leading to instability of the walls.

5.2.9 CONSTRUCTION DEWATERING

As mentioned in the previous section, the need for onsite construction dewatering should be anticipated. High groundwater flows, either through the base of the excavation or through voids in the interlocking sheet piles or the timber lagging, could lead to loss of ground resulting in reduced excavation stability.

Construction dewatering can generally be performed by pumping the water from inside and/or outside of the excavation. Generally, pumping of water from outside of the excavation is a safer approach than pumping the water from inside of the excavation. An external dewatering system may consist of the installation of perimeter dewatering wells surrounding the excavation. Prior to implementation of the external dewatering system, a pump test is highly recommended to determine the permeability of the in-situ ground and to evaluate the effectiveness of a potential external dewatering system. Typically, the design and operation of the dewatering system would be the responsibility of the construction contractor, with review and approvals from the engineering design team.

It should be noted that the groundwater level inside of the excavation should be kept at a minimum of 0.5 m below the base of the excavation to allow for development of a clean, dry and stable subgrade for construction of foundations. In addition, even where an external dewatering system is implemented, there may be potential of slight water seeping into the excavation. If this occurs, or where redundancy is needed, an internal dewatering system (i.e. pumping water inside of the excavation) should also be implemented. The internal dewatering system should be used to control potential water flow into the excavation to preserve the stability of the excavation and reduce the potential for groundwater accumulation within the excavation. The internal dewatering system may comprise collection trenches/pits and sump pits, with appropriate filtering.

Due to potential water issues, a temporary shoring system that consists of tightly spaced or interlocked pile walls systems, such as the steel sheet piles or steel sleeve will be of advantage.

It is expected that the bearing surface will consist glacial clay till. As a result, the bearing surface may not be too susceptible to disturbance. However, avoiding disturbance of the bearing surface is still vital. Protection of the bearing surface may be achieved with the placement of a lean-mix concrete slab (or mudslab) directly on the approved bearing subgrade.

Groundwater discharge should meet the necessary local government requirements for water quality and should be designed to facilitate sampling if and where required. In this regard, where fine particles are collected within the groundwater, it may be necessary to remove the fines prior to disposal off site. This may require the use of silt curtains, sedimentation or filtering to contain suspended water-born particles and limit sediment transport during discharge. Furthermore, the loss of fine particles may be an indication of a more serious concern regarding the potential for piping. Therefore, the loss of ground both from the excavation base and from behind the shoring should be monitored during construction. It is recommended that the condition of the excavation base be evaluated by WSP during construction to determine the effectiveness of the external and internal dewatering system as well as to assess the bearing surface conditions.

5.2.10 SOLDIER PILES AND TIMBER LAGGING SYSTEM

It is understood that typically, a solider pile and timber lagging system is preferred by contractors. The solider pile and timber lagging system is only considered to be possible, if the following minimum conditions are met:

1. Dewater groundwater adjacent to and below the lift station excavation to a minimum 0.5 m below the base of the excavation or alternatively conduct excavation at time of year when natural groundwater conditions meet this condition; and
2. Control water seepage through the excavated walls and loss of ground below and behind the timber lagging.

If the above minimum conditions are met, it is expected that solider and timber lagging shoring can be utilized. Typically, for shallow excavations, a cantilevered system would be used; however, given the depth of the excavation, internal bracing might be needed.

Lateral earth pressures for the design of the solider pile and timber lagging shoring system design can be obtained from Section 5.2.6. It should be noted that the passive earth pressure for the solider piles below the excavation base should be applied to the flange width of the piles.

Generally, H-piles are utilized as soldier piles and are installed in one of two manners:

1. Driven to refusal in the glacial clay till or underlying bedrock;
2. Drilled hole with H-piles concreted in place in the till, such that the flat faces of the H-piles are directed toward the interior of the excavation.

During the pile installation, vibrations created during pile driving may affect the nearby existing structures. The effect of vibration can be reduced by pre-drilling the pile hole (i.e. to about 6 m) as opposed to driving the pile right from the ground surface.

If solider piles are installed in a drilled hole, the contractor should use a protective steel casing to maintain the pile holes in an open and dry condition. Where seepage cannot be controlled, all concrete will have to be placed using tremie methods. In addition, the glacial clay till might contain cobbles and boulders and therefore drilling of the steel soldier piles holes may require the removal of these obstructions.

Following installation of the piles, the soil in front of (i.e. on the interior of the proposed excavation) and immediately between the piles is excavated in a staged manner to ensure stability of the shoring system. During each stage, the timber lagging boards are placed between the pile flanges and bolted as required. At pre-determined depths steel anchors or struts can be installed. To prevent seepage and soil migration through the small gaps between the lagging boards, a non-woven geotextile should be installed behind the wood lagging.

5.2.11 STEEL SHEET PILES

Where steel sheet piles are used, the application conditions for solider pile and timber lagging system (Section 5.2.10) are also applied for sheet piled wall except that lowering the exterior groundwater level is not required provided that the piles are extended to the required depth below the excavation base or to refusal in the glacial clay till or underlying bedrock. Given the depth of the excavation, internal bracing might be needed.

Design and construction method of installation of sheet piled wall shall be completed by contractors. Lateral earth pressures for the design of the sheet piled wall can be obtained from Section 5.2.6. It should be noted that the glacial clay till might contain cobbles and boulders and may impact to advancing sheet piles.

Following installation of the piles, the soil in front of (i.e. on the interior of the proposed excavation) and immediately between the piles is excavated in a staged manner to ensure stability of the shoring system. At pre-determined depths steel anchors or struts can be installed.

5.2.12 EXCAVATION STAGING

All shoring members (i.e. struts, walers, timber lagging, sheet piles, soldier piles and etc.) should be designed and checked or all stages of partial and full excavation.

5.2.13 SHORING WALL MONITORING

Shoring performance and general condition of the excavation should be monitored both during and following construction of the shoring wall. The shoring wall should be regularly monitored for ground loss and the presence of voids behind the shoring, particularly where seepage is encountered during excavation. All voids detected should be immediately backfilled with sand and/or grout. Shoring monitoring should include measurement of lateral and vertical movement of shoring walls, settlement monitoring of hard surfaced areas around the site as applicable, and measurement of vertical movements of the excavation base.

For sheet piled walls, the lateral wall movement is anticipated to be less than two (2) percent of the excavation depth throughout all stages of construction, although movements will depend on the rigidity of the design, as the lateral wall movement of the sheet piled walls is a function of the relative stiffness of the sheet piles and the spacing of the lateral support (i.e. struts). Movements will also depend on the workmanship, and how quickly the lateral support can be provided during the excavation. These movements will generally be smaller if the horizontal supports are installed as soon as the support level is reached. Similarly, vertical settlement of surface grades within a horizontal distance of the shoring equal to three times the depth of the excavation and is anticipated to be less than one (1) percent of the depth of excavation if construction is in keeping with best practices. WSP can provide further guidance on the excavation movements, once the detail of the shoring design is finalized. If greater lateral movements or vertical settlements are observed, the design and construction of the shoring system should be reviewed.

5.2.14 OTHER CONSIDERATIONS

It should be noted that removal of the sheet piles, soldier piles, wood lagging, etc. after construction will create voids in the soil behind the walls of the lift station. All voids should be properly backfilled with either granular fill, compacted in place by water jetting, or using a cement grout. The choice of backfill material should take into account designs for both horizontal stresses and frost effects pertinent to the specific backfill type selected. As an alternative, voids may be eliminated by casting the lift station walls directly against the steel sheet piles and leaving the steel sheets in place permanently, if sheet piles are utilized.

5.3 LIFT STATION FOUNDATION

5.3.1 DESIGN FOOTING BEARING PRESSURE

The lift station will consist of a shaft supported by a raft foundation at a minimum depth of 6.0 m below ground surface. On this basis, the ultimate bearing capacity of the glacial clay till, which will form the foundation for the concrete footing at the site may be taken as 1,000 kPa. A geotechnical resistance factor of 0.5 should be applied to the ultimate bearing capacity and compared to the factored loads under the limit state design approach. As a result, the proposed footing will have a factored geotechnical resistance of 500 kPa.

The bearing surface of the lift station should be excavated in a manner to minimize disturbance of the subgrade. The bearing surface should be trimmed free of softened or loose soil, kept free of water, and protected from any other environmental effects that will cause disturbance to the subgrade condition (such as frost).

Generally, settlement of the lift station is expected to be minimal. It should be noted that additional settlement could occur where disturbance and/or softening of the bearing surface occurs during construction. A small amount of heave of the excavation base will occur during excavation followed by elastic settlement of the bearing surface as construction proceeds. The magnitude of the settlement will depend on the applied net pressure. WSP can provide settlement estimates once the applied pressure is known.

5.3.2 BUOYANCY

Based on the anticipated groundwater level at the Site, the lift station will be subject to uplift pressure due to buoyancy. For design purposes, the buoyancy force may be estimated assuming a groundwater table at surface. Resistance to buoyancy will be provided by the dead weight of the lift station and soil friction along the exterior sidewalls of the lift station. Uplift resistance may also be provided by extending the raft beyond the walls of the lift station to create a lip that is retrained by backfill that is placed over it. In case the shoring system will be removed, the factored side friction resistance along the perimeter walls of the lift station between the soil and the concrete may be taken as 15 kPa below a depth of 2.5 m. If sheet piles will be utilized and the lift station walls will be casted directly against the steel sheet piles and leaving the steel sheets in place permanently, the factored side friction resistance may be taken as 40 kPa below a depth of 2.5 m. Where permanent sheet piles are used, the lift station wall will need to be structurally connected to sheet piles.

5.3.3 LATERAL EARTH PRESSURES ON BURIED LIFT STATION WALLS

The permanent walls of a buried concrete lift station will be required to resist lateral earth pressures and hydrostatic pressure from the surrounding soil and groundwater in the long term. Where the lift station is cast directly against the temporary shoring or where backfill that is placed against the wall of the chamber is lightly compacted, the lateral soil pressure (p) distribution may be assumed to be trapezoidal in shape and increase linearly with depth as illustrated on Appendix D.

Lightly to moderately compacted backfill typically corresponds to soils placed and compacted to between 93 percent and 95 percent of SPMDD. Settlements under the self-weight of such compacted backfill is dependent on the soil type used, however usually would not exceed 2 percent of the fill height. In cases where the backfill is well to highly compacted, settlements will be less, however, the additional lateral pressures induced on the wall by compaction must also be considered in the design of the below grade walls. WSP can provide lateral earth pressure distributions for highly compacted backfill on request.

The design of the lift station wall should also take into account the hydrostatic component acting on the wall. The groundwater levels considered in design of the subsurface walls should be taken as at surface for this Site.

It is anticipated that a braced excavation will be formed against the face of the excavation, and as such, limited relaxation of the retained soils will occur. As such, the use of the 'at-rest' lateral earth pressure coefficient K_o in the design of unyielding lift station walls is recommended. The 'at-rest' earth pressure coefficient is presented in Table 5-1 in Section 5.2.6.

It is recommended that a cap of clay, concrete or asphalt should be placed at or just below the ground surface adjacent to the foundation walls to reduce the migration of surface water into the underlying granular backfill materials between the walls of the lift station and native soils. If a clay cap is used, the clay cap should have a minimum thickness of approximately 0.3 m and should extend a minimum of 3 m horizontally from the uplift station walls.

5.3.4 UPLIFT DUE TO FROST

Resistance to the adfreeze stress would be provided through the combined mass of the lift station structure plus frictional resistance of the soil in contact with the concrete walls below the depth of frost. The allowable frictional resistance between the soil and the concrete may be taken as recommended in Section 5.3.2. Alternatively, the effect of adfreeze can be reduced through the application of a bond breaker around the perimeter of the lift station within the depth of frost. A suitable bond breaker may consist of a Dow Ethafoam product or a smooth geosynthetic liner material fixed to the exterior of the chamber walls.

Notwithstanding the above, the lift station will extend through the zone of frost penetration. Portions of the lift station located within the depth of frost penetration must be structurally designed to resist increased lateral pressures induced by frost. In the case of unyielding walls exposed to frost penetration above the groundwater table, it is recommended that $K_o = 1.0$, be used to account for lateral frost pressures[2].

It should be noted that uplift due to frost and uplift due to buoyancy result from different mechanisms and they occur during different seasons. On this basis, these conditions are not additive and should be addressed separately in design.

Other frost considerations are provided in Section 7.

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[2] As per Canadian Foundation Engineering Manual, 3rd Edition, P. 429, an earth pressure coefficient K=1 should be used in combination with insulation for highly frost susceptible soils.

6 DESIGN AND CONSTRUCTION RECOMMENDATIONS FOR THE EXISTING LAGOON UPGRADES

6.1 GENERAL DISCUSSION

Soil conditions encountered immediately south of the existing lagoon where the upgrades are proposed to occur (test holes TH24-03 to TH24-08) consisted of layers of medium plastic clay, silt, sand extending to depths ranging from 3.7 m to 6.4 m below ground surface, underlain by medium plastic clay till. The medium plastic clay was absent in some locations or extended to 1.2 m to 2.1 m below ground surface. Seepage and sloughing were recorded during drilling within shallow silt and sand layers. The most recent groundwater level measured on August 28, 2024, in the piezometer installed in TH24-03 indicates a groundwater level of 1.68 m below existing ground surface.

It is understood that the upgrades of the existing lagoon involve relining the existing secondary cell, raising the berms and decommissioning the existing primary cell. The lagoon facility will be expanded to the south with SAGR cells, an operations building, and potentially a new aerated cell. The lagoon dyke slope will be 4H:1V or flatter. It is also understood that clay liner is preferable if the clay borrow source(s) satisfies the permeability requirement. Other details (e.g. width and elevation of the top of the dykes, elevation of lagoon bottom, normal operation level, etc.) are not available at the time of writing this report.

Per the scope of work required in the proposal WPG2024.014, considerations and recommendations will be provided for design and construction of the dykes of the lagoon and the potential aerated cell, and foundation of the operations building.

WSP assumed that the upgrades (i.e. raising the dykes) or the dykes of potential aerated cell will be constructed using local clay material exploited from one of the proposed borrow sources investigated. WSP also anticipated that a thickened edge slab is a preferent foundation option for the operations building. Recommendation for other foundation types can be provided upon request.

6.2 LAGOON

6.2.1 LAGOON DYKE

The subsurface conditions encountered in the site are anticipated to provide a suitable foundation for the proposed dykes subject to subgrade treatment. The recommended subgrade treatment includes clearing and stripping all organics and topsoil. The exposed subgrade, if found suitable, should be scarified and re-compacted to at least 95 percent of SPMDD. Soils with organic matter, soft or week zones should be excavated and replaced with compacted local clay material. The local clay material can be used to replace detrimental zones and construct the dykes (Section 10).

Clay dykes not exceeding 3 m in height may typically be designed with side slopes not exceeding 5H:1V. Detailed slope stability analysis is required for dykes greater than 3 m in height or steeper than 5H:1V side slopes. The clay should be placed in layers not to exceed 200 mm non-compacted thickness at moisture content within ±2 percent of the optimum moisture content and compacted to at least 95 percent of SPMDD. The standard proctor tests were conducted for two samples of the local clay material. The results are provided in Table 4-2. More standard proctor tests may be recommended prior to the construction depending on the exploitation areas and variability of the clay. The required compaction moisture content variation might need to be adjusted depending on these results.

Erosion protection measures will be required on the slope surfaces of the proposed dykes. The exterior slopes can be protected using a suitable vegetation cover. A rip-rap protection layer can be used on the interior slopes to provide protection against rainfall, snowmelt, wave action, or any other erosive actions. Further recommendations can be provided as part of the detailed design phase.

6.2.2 LINER

Given the soil condition encountered (Section 4.1.2), the anticipated lagoon subgrade did not consistently consist of suitable clay that could be considered as a clay liner. Therefore, a clay liner constructed of suitable compacted clay obtained from the borrow source is required, or alternatively a synthetic liner should be considered.

The clay material in the explored site NW-14-3-20W is considered suitable for clay liner (Section 10). The clay liner should be minimum 1.0 m thick, placed in uncompacted lift of maximum 200 mm, conditioned the moisture to ±2% of optimum moisture content, and compacted to minimum 95 % SPMDD.

6.2.3 SUBGRADE PREPARATION

Lagoon dyke and liner may be constructed adopting the following recommendations:

1. All topsoil, organic clay should be stripped from the footprints of the dykes. The stripped material should be stockpiled separately for future use to cover the side slopes and protect against erosion. Organic soil may be defined as any soil containing greater than 6% organic content by weight. Based on visual inspection, and the organic clay encountered, the shallow soil observed was estimated to have a high enough organic content to warrant removal and a removal depth of 0.3 m to 0.6 m is anticipated. WSP can undertake testing to determine organic content on request during construction if the organic content of the subgrade soil is in question. It is important to note that the excavation depth within the lagoon bottom will depend on the unsuitable material revealed at the time of construction and the thickness of the clay liner where clay liner is applied.
2. Once final subgrade is achieved, the subgrade should be evaluated by qualified personnel to detect soft or weak areas, and to verify that soils are as expected and that no unsuitable materials remain.
3. Provided the subgrade can support heavy equipment without excessive disturbance, and where practical, the subgrade should be proof rolled using a fully loaded tandem dump truck to identify soft, weak or compressible areas. The suitability of the subgrade for proof rolling should be assessed by the geotechnical engineer at the time of construction.
4. The subgrade should be protected from frost, desiccation, inundation and excessive wheel loads at all times. Subgrade preparation and fill placement under freezing conditions should be avoided. The use of frozen soils for fill, placement/compaction of frozen soils, and placement/compaction of soils over frozen subgrade, should also be avoided. If the subgrade is frozen, it must be thawed and potentially repaired before placing fill, allowing time for thaw-settlements to occur. Rolling the subgrade after thawing is complete may be suitable to mitigate thaw-related settlement potential, however may not mitigate the full depth of disturbed soil.
5. In case of upgrading the existing dykes, after stripping all topsoil and unsuitable material, the existing dykes should be benched prior to placing and compacting new materials to provide better bond between the extension and the existing dyke.

6.3 GRADE-SUPPORTED THICKNED EDGE SLAB FOR OPERATIONS BUILDING FOUNDATION

6.3.1 GENERAL DISCUSSION

Shallow soil conditions at the site are considered feasible for a grade supported thickened slab. However, given the soil conditions encountered, it is anticipated that suitable and uniform subgrade materials might be not available. Therefore, development of a uniform layer to support the thickened edge slab will be required. In this regard, the organic clay was noted to extend to 0.6 m below grade and should entirely be removed and replace with engineered clay fill. Construction on silt is expected to be complicated, and where silt or otherwise unsuitably soft subgrade conditions are encountered at or below the subgrade elevation, full-depth excavation of the silt layer (if practical) or sub-excavation and placement of a rock bridging layer with a geotextile separator will be necessary to allow placement and compaction of structural fill.

It should be recognized that as with all grade supported slabs, differential movements due to non-uniform subgrade support could occur. At this site, the potential for frost heave to occur may be significant given the soil type and the depth to groundwater encountered. Measures to reduce the effects of frost heave below unheated slabs could include replacing all or a portion of the frost-susceptible soils within the active zone (i.e. 2.5 m) with free-draining, non-frost susceptible material such as gravel; using insulation below and around the perimeter of the slab to provide the equivalent of 2.5 m of soil cover; or providing drainage to maintain the groundwater level below the frost zone.

In addition, grade supported slab foundation are subject to heave where seasonal frost penetrates below the slab although the risk is not considered to be a significant to the interior slab in heated structures unless the slab foundation will be subjected to freezing conditions during construction. If the subgrade is allowed to freeze after concrete have been placed, some differential movements are expected to occur, potentially resulting in mid-term to long-term movements. Slab foundation for spaces that are planned to be heated should not be allowed to freeze once the concrete has been placed.

Assuming that the risk of grade supported slab movements discussed above is acceptable to the owner, and provided adequate controls are put in place to prepare and protect subgrade conditions and to reduce the potential access of the subgrade soils to free water, then grade-supported concrete slabs can be designed and constructed as recommended herein. If the potential for total and differential slab movement is intolerable, then a structurally supported slab should be used.

Essentially, a thickened edge slab consists of a grade-supported slab that is poured monolithically with stiffened footing sections (i.e. spread footings) that support concentrated loading applied to the slab such as the structure envelope, column loads, etc. The un-thickened portions of the slab should be designed and constructed using the recommendations for grade supported slabs outlined in Section 6.3.2. The thickened edges and/or internal beams of stiffened slabs should be designed and constructed using the recommendations outlined in Section 6.3.3.

6.3.2 GRADE-SUPPORTED CONCRETE SLABS

Grade-supported concrete slab should be underlain by a minimum gravel structure thickness of 300 mm consisting of 100 mm of crushed gravel base course compacted to 100% of SPMDD at ±3 percent of optimum moisture content, underlain by 200 mm of crushed subbase compacted to 98% of SPMDD at ±3 percent of optimum moisture content. It is recommended that the gravel base course and gravel subbase materials should meet the material and gradation requirements for Class "A" and Class "C" gravel, respectively, outlined in the latest edition of Manitoba Infrastructure Standard Construction Specification. Other gradations may be suitable but should be reviewed by the geotechnical engineer prior to use.

For the purposes of concrete slab thicknesses and reinforcement design (i.e. rigidity), grade-supported concrete slabs designed on the minimum recommended gravel structure above, and constructed on an approved subgrade prepared as outlined in Section 6.3.4 below may be designed assuming a Westergaard subgrade reaction modulus (k) of 35 MPa/m. If additional subgrade stiffness is required, WSP can provide recommendations for additional gravel structure thickness on request.

To reduce the effects of slab movements on the building structure, the following provisions are recommended:

1. Design equipment and partition walls bearing on the slab with a void space to minimize the potential for structural damage if the slab heaves.
2. Provide control joints at regular intervals in the slab to reduce random cracking.
3. Construct the floor independent of structural elements using isolation joints.
4. Due to the potential for frost heaving of exterior slabs, all sidewalks and apron slabs should be structurally separate from the structure and should not be dowelled into the grade beam or the interior slabs, except at doorways.
5. Where it is proposed to dowel exterior slabs into structure components, or where frost related movement of the slab is undesirable, rigid insulation could be placed on the subgrade and below the slab concrete and granular structure, to reduce the effects of frost penetration beneath the slab. Placement of vertical insulation along the vertical sides of grade beams should be avoided to allow beneficial heat loss from the building and lessen frost effects.
6. A polyethylene vapour barrier may be utilized directly below floor slabs to limit moisture migration through the slab. It should be noted that curing problems (delays before final finishing), curling of the slab at the edges and shrinkage cracking might be encountered where the concrete slab is cast directly on the poly. Where the concrete will not require a finished floor covering, a vapour barrier is not necessarily required.

6.3.3 THICKENED EDGES AND INTERNAL BEAMS

The thickened edges and/or internal beams of stiffened slabs bearing on the native, undisturbed, firm to stiff clay or firm silt may be designed as spread footings using a factored ultimate bearing pressure of 90 kPa. The recommended bearing pressure assumes a minimum width of 0.6 m for all perimeter and internal stiffeners, bearing at least 0.3 m below final grade, and are subject to inspection and approval of all bearing surfaces by a qualified geotechnical engineer. It should be strictly noted that the recommended design bearing pressures have been provided assuming an Ultimate Limit State (ULS) defined by plastic soil deformation and geotechnical failure of the footing. In other words, no reduction has been applied to the bearing pressure values to maintain deformations within a zone of elastic or elastic-plastic deformation, nor to ensure a maximum level of tolerable deflection. Reduced bearing pressures may be required where the ULS of the footing is to be defined by a specified deformation of foundation subgrade that could lead to the ULS state being induced in the superstructure.

With respect to serviceability and settlement, the bearing pressure at the Serviceability Limit State (SLS) can only be determined from settlement analyses which in turn can only be evaluated once the final foundation configuration, including both depth and footing size, is known. However, conservatively, the serviceability limit state may be assumed equivalent with traditional working stress design, and in this regard, the bearing pressure at the serviceability limit state for thickened sections bearing embedded at least 0.3 m below final grade, and a maximum of 1.0 m wide, may be taken equivalent to factored ULS pressure, 90 kPa. The elastic settlement potential of footings up to 1 m in width is estimated to be in the range of 25 mm or more for a sustained loading of 90 kPa, with settlements expected to occur almost immediately upon loading of the bearing surface. In addition, clay layer up to 2.0m thick (TH24-05) will potentially cause some consolidation settlement for the footing in long term.

WSP can provide a recommended gravel configuration, or revised bearing values, for specific footing sizes and serviceability tolerances upon request.

6.3.4 SUBGRADE PREPARATION AND CONSTRUCTION RECOMMENDATIONS

Grade-supported slab with thickened edges may be constructed adopting the following recommendations:

1 Excavate to design subgrade elevation, which should be taken as the top of the slab minus the thickness of the slab provided in Section 6.3.2. Further remove any unsuitable and/or deleterious materials if discovered during construction including silt, organic soils, etc. Organic soil may be defined as any soil containing greater than 6% organic content by weight.

2 Fill materials required to raise grades to the underside of the granular section described above would ideally consist of additional 50 mm max. crushed limestone sub-base (i.e. GSB-C in accordance with the Specification 901(i) – Material Specification for Aggregate – Granular Course, Manitoba Transportation and Infrastructure), placed in maximum 150 mm thick lifts and uniformly compacted to 98% of SPMDD. The use of clay fill, or bridging material could also be considered. Where clay fill is used to raise grades, the clay fill should be uniformly moisture conditioned to 0 to 2% above optimum moisture content depending on the specific characteristics of the clay, and uniformly compacted to 95% of SPMDD, in lifts of maximum of 200 mm thick as measured uncompacted.

3 Other recommendations outlined in Section 6.2.3.

7 FROST DESIGN CONSIDERATIONS

7.1 FROST PENETRATION DEPTH

The upper stratigraphy at the Site is considered highly frost susceptible. As such, frost effects should be considered for foundations or surface structures sensitive to movement. Based on historical temperature data for the area, a design frost penetration of 2.5 m below final grade is recommended in areas that will not have regular snow or vegetative ground cover. It should be noted that this recommended frost penetration depth extends both vertically and laterally behind final surfaces.

7.2 FROST FORCES

Foundation elements supporting unheated structures and perimeter foundation elements supporting heated structures should be designed to resist the frost forces discussed herein. Similarly, interior piles and other foundation elements for heated structures should be designed for the unheated condition if they will be exposed to freezing conditions during construction. Potential frost forces acting on foundation elements include adfreeze pressures along the sides of foundation elements (i.e. grade beams, footings, thickened slabs, lift station etc.) that extend through the frost zone, as well as frost heave on the undersides of such elements or connecting supports located within the anticipated depth of frost penetration.

Adfreeze forces acting on buried structures and foundation elements may be determined assuming an unfactored unit adfreeze stress of 65 kPa applied only to the exterior surface area of the portion of the structure or foundation element located within the zone of frost penetration. A load factor of 1.25 should be applied to obtain the factored adfreeze stress. The resistance to frost uplift forces will be provided by the skin friction of that portion of piles extending below the depth of frost penetration using values provided above for resistance to tensile loads. A geotechnical resistance factor of $\phi = 0.75$ should be applied to the unfactored frost uplift resistance to determine the factored uplift resistance. The adfreeze stress could be reduced by affixing a 'bond-break' or 'friction reducer', such as greased poly-wrap or geosynthetic liner material, to portions of the structure or foundation element located within the zone of frost penetration.

With respect to frost heave, the potential for frost heave pressures to develop on the underside of foundation elements should be mitigated by one or more of the following optional measures:

1. Foundation elements such as grade beams and pile caps may be designed to extend below the frost penetration depth. Alternatively, insulation may be used to establish a minimum equivalent depth of soil cover. WSP can provide a recommended insulation configuration upon request.
2. Where the undersides of foundation elements are located above the depth of frost penetration, a void-forming product should be installed beneath the undersides of the grade beams, pile caps, and any other connecting elements located within the depth of frost penetration above the groundwater table. The recommended minimum thickness of the void is 150 mm. Alternatively a compressible material may be used in lieu of a void forming material, and the uplift pressures may be taken as the crushing strength of the compressible medium. It is recommended that a frost heave potential of 150 mm be assumed in determining the required thickness for the void-filler and the associated uplift pressures associated with the thickness used.
3. The finished grade adjacent to all structures and foundation elements should be capped with well compacted clay and sloped away from the structure so that the surface runoff is not allowed to infiltrate and collect in void spaces or saturate the compressible medium. Saturation of the void and/or compressible medium with water will negate the function of the void or compressible medium, if it becomes frozen, and therefore saturation should be avoided.

8 FOUNDATION CONCRETE

Where concrete elements outlined in this report and all other concrete in contact with the local soil will be subjected in service to weathering, sulphate attack, a corrosive environment, or saturated conditions, the concrete should be designed, specified, and constructed in accordance with concrete exposure classifications outlined in the latest edition of CSA standard A23.1, Concrete Materials and Methods of Concrete Construction. In addition, all concrete must be supplied in accordance with current Manitoba and National Building Code requirements.

Chemical tests (i.e. Water Soluble Sulphate in Soil Samples) do not include in the scope of work for this project. Therefore, the degree of sulphate exposure is not evaluated for this Site. If the tests will be completed prior to the construction and the results show that the water soluble sulphate concentrations in the soil are of 0.2% or higher; or the local practices reveal that the use of the sulphate resistance cement is common in the area, the use of sulphate resistance cement (Type HS or HSb) will be required. Furthermore, air entrainment should be incorporated into any concrete elements that are exposed to freeze-thaw to enhance its durability.

It should be recognized that there may be structural and other considerations, which may necessitate additional requirements for subsurface concrete mix design.

9 FINAL SITE GRADING, SURFACE DRAINAGE, AND SUBDRAINAGE

Sufficient gradients should be provided to promote surface drainage away from structures to reduce the potential for moisture percolation to the foundation elements. Site grading should provide positive drainage away from structures at a minimum gradient of 4% for landscaped areas within 3 m of the perimeter of the building; and at a minimum gradient of 2% for all pavement areas as well as landscape areas outside of 3 m of the building perimeters. All downspouts from the roof of the structures should be discharged away from the building and proper measures (i.e. splashguards) should be provided where necessary to reduce the potential for erosion and ponding water at the perimeter of the structures.

Excavations at the perimeter of the structure (grade beams) should be backfilled with moderately to well compacted fill and topped with a clay cap a minimum of 0.3 m thick to reduce the potential for surface water infiltration into the slab subgrade or backfill against grade beams. As a recommended minimum, the clay cap in landscape areas along the perimeter of the foundations should extend a minimum of 3.0 m from the foundation perimeters. Where pavement and/or concrete slabs meet the structure, these should be sealed against abutting structural components with a flexible seal, such as an asphaltic bead, to reduce the potential for surface water infiltration. Site grades should be maintained over the long term, with low spots being filled to maintain positive drainage away from structures.

Where a below-grade void space or crawlspace is used, sub-drainage should consist of a perimeter drain system to collect groundwater accumulation along foundation elements, and to divert ingress of surface runoff. An interior drainage collection system consisting of a minimum of one central collection line should be installed within the void space area to collect potential seepage into the area. The interior drainage collection system should be independent of the perimeter drain system.

Perimeter and internal drains should consist of perforated drains surrounded by a clean, filter wrapped, granular drain material and should be directed to one or more positive outlets such as a central collection sump(s); or should be directed by gravity flow directly into an acceptable land drainage system. Drainage lines passing through perimeter walls, or connecting to outlets such as sump or sewer, should consist of solid pipe. Depending on final elevations, site configuration, grading, or presence of a void space, it may be necessary to install several interior filter wrapped drain lines and/or drainage outlets (i.e. sumps) to control slope lengths and drainage line lengths.

10 CLAY BORROW MATERIAL

10.1 GENERAL DISCUSSIONS

As noted, two sites were investigated for potential clay borrow. The investigation revealed that only site NW14-3-20W is considered potential given the available potential clay thickness. The investigation determined a potential area of approximately 80,000 sqm to the northwest of the NW14-3-20W borrow source where test holes TH24-09 to TH24-12 were drilled. In this area, potential clay material encountered in those four test holes from ground surface or beneath a thin layer of organic clay or locally sand fill and extended to depths ranging from 3.2 m to more than 4.6 m (termination depth). A sand layer, up to 0.5m thick, was encountered below the clay and underlain by clay till. Based on the clay thickness of test holes TH24-09 to TH24-12, a potential quantity of 200,000 cum might be estimated.

Groundwater and/or sloughing were observed in the sand fill (locally) or in the sand below the clay. No groundwater seepage or sloughing was observed in the clay layer. Therefore, exploitation is expected to be favorable subject to the recommendations outlined in Section 10.3. However, the investigation indicated that the soil condition was variable in the area; sand or silt material were detected from ground surface and extended to the termination depth, 4.6 m, in numerous test holes completed, including other test holes of the NW14-3-20W. It is therefore important to note that sand and silt material and seepage within those should be expected, and dewatering plan should be prepared for the material excavation; For the same reason, it is recommended that the estimated clay borrow volume be reduced by at least 30% to account for potential soil wastage due to sand/silt layers within the clay.

It is also noticed that the test holes were 250 m approx. apart might not fully capture the soil condition since the soil condition was considered inhomogeneous within the area investigated. Furthermore, the availability of qualified local clay materials is considered a key factor for the design, particularly for selection of the liner solutions. For those reasons, it is highly recommended to advance more test holes prior to the construction to confirm the required quantity of the clay material for the construction.

10.2 QUALITY EVALUATION

Following preliminary evaluation of the potential area as noted above, a testing program was completed including Atterberg Limits, Hydrometer, standard proctors and permeability tests to classify the clay and evaluate the quality. The permeability tests were conducted on remolded specimens prepared at 95% of SPMDD corresponding to the compaction requirement of the dykes and clay liner construction. Summary of the testing results are provided in Table 4-1 and Table 4-2.

It is understood that suitable clay should have permeability $k<10^{-7}$ cm/sec. According to the test results, the potential clay is classified as medium to high plastic and the permeability values ranges from 4.76×10^{-8} to 5.92×10^{-8} cm/sec. Therefore, the clay is considered suitable for construction of the lagoon.

10.3 EXCAVATION RECOMMENDATIONS

Following recommendations are adopted for the excavation of the clay material at the borrow site.

1. Excavated material should be selective to avoid any organic, silt, sand, or deleterious material, and placement should be supervised by qualified geotechnical personnel to assess material suitability as berm and liner fill. As such, all unsuitable materials should be stockpiled separately from the clay borrow material.
2. Organic clay and sand fill should be removed. The topsoil/organic clay should be stripped and stockpiled separately for future use, potentially to cover the side slopes of the exterior side of the lagoon berms to protect against erosion.

3 Based on the test holes completed, it is recommended that the excavation to be limited to 3.0 m below existing ground surface to avoid reaching the native sand and potential groundwater seepage within the sand. It is important to note that the excavation depth should be confirmed and might be adjusted after having more test holes advanced as recommended (Section 10.1).

4 In developing the borrow area(s), all excavations should be completed in accordance with Manitoba Workplace Health and Safety Regulations. Cut slopes may have to be adjusted in the field as excavation progresses, depending upon conditions encountered (i.e. soil texture, water content and length of time that the excavation is left open). The condition of the soil around the excavation should be carefully observed to protect against development of slope instabilities or sloughing conditions. Continuous inspection is recommended since slumps or slides could occur suddenly. Seepage may occur from the silt and sand layers, particularly during or shortly after precipitation events. Inflows of any groundwater may be handled by grading the bottom of the excavation into a sump and using pumps to remove water.

5 Spoil material should not be piled closer to the excavation edge than a distance equal to the depth of the excavation. It is recommended that workers not be around the excavation during rainfall and that excavation walls be carefully inspected after rainfall for potential instabilities before work continues in the excavation.

6 Restore the excavated borrow areas, disturbed areas and associated haul roads according to the requirements of the project/owner.

11 CONSTRUCTION MONITORING AND TESTING

In accordance with Section 4.2.2.3 Field Review of the 2020 NBCC, all engineering design recommendations presented in this report are based on the assumption that an adequate level of testing and monitoring will be provided during construction by either the designer or other suitably qualified personnel. Furthermore, it is assumed that all construction will be carried out by a suitably qualified contractor experienced in foundation and earthworks construction. An adequate level of testing and monitoring is considered to be:

1. for earthworks:
   - full-time monitoring of fill quality and subgrade conditions and compaction testing.
2. for lift station foundations:
   - review of foundation design prior to construction; and
   - full-time monitoring during construction (i.e. movement of shoring system, excavation base, seepage and groundwater, etc.).
3. for concrete construction:
   - testing of plastic and hardened concrete in accordance with the latest editions of CSA A23.1 and A23.2; and
   - review of concrete supplier’s mix designs for conformance with prescribed and/or performance concrete specifications.

On the basis of the above, and given WSP’s familiarity with the subsurface conditions at this site as the Geotechnical Engineer of Record, WSP requests the opportunity to review the design drawings, and the installation of the foundations, to confirm that the geotechnical recommendations have been correctly interpreted. WSP would be pleased to provide any further information that may be needed during design and to advise on the geotechnical aspects of specifications for inclusion in contract documents. WSP can provide design modifications as required at the time of construction should subsurface conditions be found to vary from those described herein.

12 CLOSURE

The findings and recommendations presented in this report were based on geotechnical evaluation of the subsurface conditions and groundwater data observed during the site investigation described in this report. If conditions other than those reported in this report are noted during subsequent phases of the project, or if the assumptions stated herein are not in keeping with the current and/or future design stage, this office should be notified immediately in order that the recommendations can be verified and revised as required. Recommendations presented herein may not be valid if an adequate level of inspection is not provided during construction, or if relevant building code requirements are not met.

The site investigation conducted and described in this report was for the sole purpose of identifying geotechnical conditions at the project Site. Although no environmental issues were identified during the fieldwork, this does not indicate that no such issues exist. If the owner or other parties have any concern regarding the presence of environmental issues, then an appropriate level environmental assessment should be conducted.

Soil conditions, by their nature, can be highly variable across a site. The placement of fill and prior construction activities on a site can contribute to the variability especially in near surface soil conditions. A contingency should always be included in any construction budget to allow for the possibility of variation in soil conditions, which may result in modification of the design and construction procedures.

This report has been prepared for the exclusive use of Manitoba Water Services Board, and their agents, for specific application to the project described in this report. The data and recommendations provided herein should not be used for any other purpose, or by any other parties, without review and written advice from WSP. Any use that a third party makes of this report, or any reliance or decisions made based on this report, are the responsibility of those parties. WSP accepts no responsibility for damages suffered by a third party as a result of decisions made or actions based on this report.

This report has been prepared in accordance with generally accepted soil and foundation engineering practices. No other warranty, either expressed or implied, is made.

Respectfully submitted,

WSP Canada Inc.

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APPENDIX

APPENDIX

A

TEST HOLE LOCATIONS PLAN

LEGEND

⌖	TEST HOLE (LIFT STATION)
◐	TEST HOLE (EXISTING LAGOON)
◓	TEST HOLE (BORROW SOURCE - NW14-03-20-W)
●	TEST HOLE (BORROW SOURCE - NW13-03-20-W)

NOTE(S)

1. SITE LOCATION AND FEATURES ARE APPROXIMATE ONLY.

REFERENCE(S)

BASE MAP TAKEN FROM GOOGLE EARTH, DATED AUGUST 2023

0 125 250
1:7,500 METRES

CLIENT
WSP CANADA INC.

CONSULTANT

YYYY-MM-DD	2024-09-04
DESIGNED	JC
PREPARED	MD
REVIEWED	JC
APPROVED	LT

PROJECT
BOISSEVAIN - MORTON WASTEWATER TREATMENT LAGOON, LIFT STATION, EXISTING LAGOON AND BORROW SOURCES, BOISSEVAIN, MANITOBA

TITLE
SITE AND TEST HOLE LOCATION PLAN

PROJECT NO.	CONTROL	REV.	FIGURE
CA0017817.1877	400-HS-0001	A	1

Path: \\corp.pbwan.net\GLB-E&I\CA\CAWNP400-WPG\data\Jobs\CA\CA0017817.1877_Boissevain Lagoon_WSP Canada Inc\CA0017817.1877_Drawings\99_PROJECTS\CA0017817.1877\02_PRODUCTION\400-HS-0001\DWG\ | File Name: CA0017817.1877_Boissevain Lagoon.dwg | Last Edited By: wds_matthews.dolores Date: 2024-09-04 Time:12:05:14 PM | Printed By: wds_matthews.dolores Date: 2024-09-04 Time:12:05:22 PM

25 mm
0
IF THIS MEASUREMENT DOES NOT MATCH WHAT IS SHOWN, THE SHEET SIZE HAS BEEN MODIFIED FROM: ANSI B

APPENDIX

B TEST HOLE LOGS

EXPLANATION OF TERMS AND SYMBOLS

The terms and symbols used on the borehole logs to summarize the results of field investigation and subsequent laboratory testing are described in these pages.

It should be noted that materials, boundaries and conditions have been established only at the borehole locations at the time of investigation and are not necessarily representative of subsurface conditions elsewhere across the site.

TEST DATA

Data obtained during the field investigation and from laboratory testing are shown at the appropriate depth interval.

Abbreviations, graphic symbols, and relevant test method designations are as follows:

*C	Consolidation test	*ST	Swelling test
D_R	Relative density	TV	Torvane shear strength
*k	Permeability coefficient	VS	Vane shear strength
*MA	Mechanical grain size analysis	w	Natural Moisture Content (ASTM D2216)
	and hydrometer test	w_l	Liquid limit (ASTM D 423)
N	Standard Penetration Test (CSA A119.1-60)	w_p	Plastic Limit (ASTM D 424)
N_d	Dynamic cone penetration test	E_f	Unit strain at failure
NP	Non plastic soil	γ	Unit weight of soil or rock
pp	Pocket penetrometer strength	γ_d	Dry unit weight of soil or rock
*q	Triaxial compression test	ρ	Density of soil or rock
q_u	Unconfined compressive strength	ρ_d	Dry Density of soil or rock
*SB	Shearbox test	C_u	Undrained shear strength
SO_4	Concentration of water-soluble sulphate	→	Seepage
		▼	Observed water level

* The results of these tests are usually reported separately

Soils are classified and described according to their engineering properties and behaviour.

The soil of each stratum is described using the Unified Soil Classification System[1] modified slightly so that an inorganic clay of "medium plasticity" is recognized.

The modifying adjectives used to define the actual or estimated percentage range by weight of minor components are consistent with the Canadian Foundation Engineering Manual[2].

Relative Density and Consistency:

Cohesionless Soils

Relative Density	SPT (N) Value
Very Loose	0-4
Loose	4-10
Compact	10-30
Dense	30-50
Very Dense	>50

Cohesive Soils

Consistency	Undrained Shear Strength c_u (kPa)	Approximate SPT (N) Value
Very Soft	0-12	0-2
Soft	12-25	2-4
Firm	25-50	4-8
Stiff	50-100	8-15
Very Stiff	100-200	15-30
Hard	>200	>30

Standard Penetration Resistance ("N" value)
The number of blows by a 63.6kg hammer dropped 760 mm to drive a 50 mm diameter open sampler attached to "A" drill rods for a distance of 300 mm after an initial penetration of 150 mm.

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[1] "Unified Soil Classification System", Technical Memorandum 36-357 prepared by Waterways Experiment Station, Vicksburg, Mississippi, Corps of Engineers, U.S. Army. Vol. 1 March 1953.

[2] "Canadian Foundation Engineering Manual", 3rd Edition, Canadian Geotechnical Society, 1992.

MODIFIED UNIFIED CLASSIFICATION SYSTEM FOR SOILS

MAJOR DIVISIONS			SYMBOLS			TYPICAL DESCRIPTION	LABORATORY CLASSIFICATION CRITERIA
			USCS	GRAPH	COLOUR		
COARSE GRAINED SOILS (MORE THAN HALF BY WEIGHT LARGER THAN 75um)	GRAVELS MORE THAN HALF THE COARSE FRACTION LARGER THAN 4.75mm	CLEAN GRAVELS (TRACE OR NO FINES)	GW	▨	RED	WELL GRADED GRAVELS, GRAVEL-SAND MIXTURES, LITTLE OR NO FINES	$C_u=D_{60}/D_{10}$ >4; $C_c=(D_{30})^2/(D_{10}xD_{60})$ = 1 to 3
			GP	▨	RED	POORLY GRADED GRAVELS, GRAVEL-SAND MIXTURES, LITTLE OR NO FINES	NOT MEETING ABOVE REQUIREMENTS
		DIRTY GRAVELS (WITH SOME OR MORE FINES)	GM	▨	YELLOW	SILTY GRAVELS, GRAVEL-SAND-SILT MIXTURES	ATTERBERG LIMITS BELOW "A" LINE OR PI LESS THAN 4
			GC	▨	YELLOW	CLAYEY GRAVELS, GRAVEL-SAND-CLAY MIXTURES	ATTERBERG LIMITS ABOVE "A" LINE AND PI MORE THAN 7
	SANDS MORE THAN HALF THE COARSE FRACTION SMALLER THAN 4.75mm	CLEAN SANDS (TRACE OR NO FINES)	SW	▨	RED	WELL GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES	$C_u=D_{60}/D_{10}$ >6; $C_c=(D_{30})^2/(D_{10}xD_{60})$ = 1 to 3
			SP	▨	RED	POORLY GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES	NOT MEETING ABOVE REQUIREMENTS
		DIRTY SANDS (WITH SOME OR MORE FINES)	SM	▨	YELLOW	SILTY SANDS, SAND-SILT MIXTURES	ATTERBERG LIMITS BELOW "A" LINE OR PI LESS THAN 4
			SC	▨	YELLOW	CLAYEY SANDS, SAND-CLAY MIXTURES	ATTERBERG LIMITS ABOVE "A" LINE AND PI MORE THAN 7
FINE-GRAINED SOILS (MORE THAN HALF BY WEIGHT SMALLER THAN 75um)	SILTS BELOW "A" LINE NEGLIGIBLE ORGANIC CONTENT	$W_L < 50\%$	ML	▨	GREEN	INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTY SANDS OF SLIGHT PLASTICITY	CLASSIFICATION IS BASED UPON PLASTICITY CHART (SEE BELOW)
		$W_L > 50\%$	MH	▨	BLUE	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS, FINE SAND OR SILTY SOILS	
	CLAYS ABOVE "A" LINE NEGLIGIBLE ORGANIC CONTENT	$W_L < 30\%$	CL	▨	GREEN	INORGANIC CLAYS OF LOW PLASTICITY, GRAVELLY, SANDY OR SILTY CLAYS, LEAN CLAYS	
		$30\% < W_L < 50\%$	CI	▨	GREEN-BLUE	INORGANIC CLAYS OF MEDIUM PLASTICITY, SILTY CLAYS	
		$W_L > 50\%$	CH	▨	BLUE	INORGANIC CLAYS OF HIGH PLASTICITY, FAT CLAYS	
	ORGANIC SILTS & CLAYS BELOW "A" LINE	$W_L < 50\%$	OL	▨	GREEN	ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY	WHENEVER THE NATURE OF THE FINES CONTENT HAS NOT BEEN DETERMINED, IT IS DESIGNATED BY THE LETTER "F", E.G. SF IS A MIXTURE OF SAND WITH SILT OR CLAY
		$W_L > 50\%$	OH	▨	BLUE	ORGANIC CLAYS OF HIGH PLASTICITY	
HIGHLY ORGANIC SOILS			PT	▨	ORANGE	PEAT AND OTHER HIGHLY ORGANIC SOILS	STRONG COLOUR OR ODOUR, AND OFTEN FIBROUS TEXTURE

SPECIAL SYMBOLS

LIMESTONE	▦	OILSAND	○
SANDSTONE	▨	SHALE	▤
SILTSTONE	▨	FILL (UNDIFFERENTIATED)	▩

SOIL COMPONENTS

FRACTION	U.S. STANDARD METRIC SIEVE SIZE		DEFINING RANGES OF PERCENT BY WEIGHT OF MINOR COMPONENTS	
GRAVEL	PASSING	RETAINED	PERCENT	DESCRIPTOR
COARSE	76mm	19mm		
FINE	19mm	4.75mm	35 - 50	AND
SAND				
COARSE	4.75mm	2.00mm	30 - 35	Y / EY
MEDIUM	2.00mm	425µm	10 - 20	SOME
FINE	425µm	75µm		
FINES (SILT OR CLAY BASED ON PLASTICITY)	75µm		1 - 10	TRACE

OVERSIZED MATERIAL

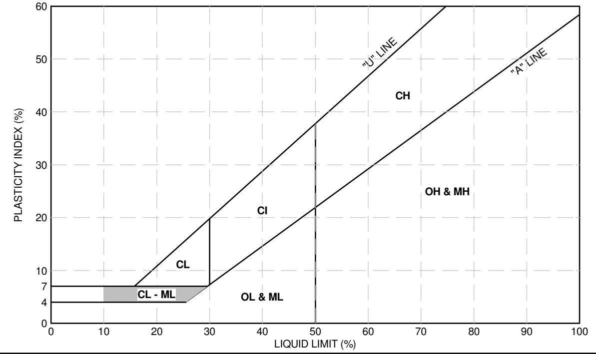
ROUNDED OR SUBROUNDED:

COBBLES 76mm to 200mm
BOULDERS > 200mm

NOT ROUNDED:

ROCK FRAGMENTS ? 76mm
ROCKS > 0.76 CUBIC METRE IN VOLUME

PLASTICITY CHART FOR
SOILS PASSING 425µm SIEVE



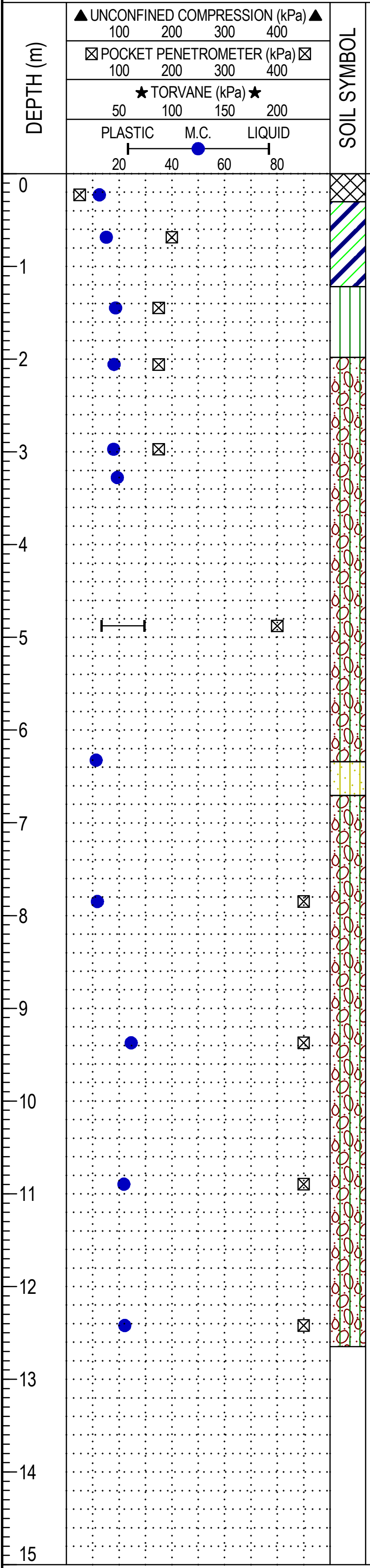
NOTES:

1. ALL SIEVE SIZES MENTIONED ARE U.S. STANDARD ASTM E.11.
2. COARSE GRAINED SOILS WITH TRACE TO SOME FINES GIVEN COMBINED GROUP SYMBOLS, E.G. GW-GC IS A WELL GRADED GRAVEL SAND MIXTURE WITH TRACE TO SOME CLAY.
3. DUAL SYMBOLS ARE USED TO INDICATE BORDERLINE SOIL CLASSIFICATIONS.

WSP Canada Inc.

PROJECT: Boissevain Lagoon	DRILLER: Paddock Drilling Ltd.	TEST HOLE ID: **TH24-01**
CLIENT: WSP Canada Inc.	DRILL RIG: Track Mounted Acker Mp5	PROJECT No: CA0017817.1877
LOCATION: Lift Station	DRILL METHOD: 125mm SSA/200mm HSA	ELEVATION: Not Surveyed

SAMPLE TYPE	▨ Shelby Tube	◿ No Recovery	☒ SPT (N)	☰ Grab Sample	▥ Split-Pen	◧ Core
BACKFILL TYPE	■ Bentonite	⁚ Pea Gravel	▨ Drill Cuttings	⁖ Grout	▥ Slough	⁘ Sand



MUSCS	SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NO	SPT (N)	COMMENTS
SP	SAND (FILL) - gravelly, some silt, trace clay, poorly graded, fine to medium grained, loose (inferred), brown, - below 0.2m, some clay	☰	1		
CI	CLAY - silty, trace sand, medium plastic, moist, stiff, greyish brown	☰	2		
ML	SILT - trace to some clay, trace sand, low plastic, moist, stiff, dark grey	☰	3		
CI	CLAY (TILL) - silty, trace sand, trace gravel, medium plastic, moist, stiff, brown	☰	4		
		☰	5		
		☒	6	10	
	- below 4.6m, grey	▨	7		
	- below 5.2m, very stiff to hard				Particle Size Analysis - Sample 7 @ 4.6m: Gravel= 0.0% Sand= 39.2% Silt= 37.6% Clay= 23.2%
SM	SAND - some gravel, some silt, trace clay, poorly graded, fine to medium grained, very moist, dense, grey	☒	8	46	
CI	CLAY (TILL) - silty, trace sand, trace gravel, medium plastic, moist, hard, grey	☒	9	43	
		☒	10	38	
		☒	11	46	
		☒	12	43	

TEST HOLE TERMINATED AT 12.6m BELOW EXISTING GRADE
Sloughing was observed between 1.2m and 2.0m during drilling.
Seepage was observed between 6.3m and 6.7m during drilling.
Test hole remained open to 11.3m with water accumulated to 8.8m prior to backfill.
Test hole backfilled with auger cuttings and bentonite.

WSP Canada Inc.

LOGGED BY: JC	COMPLETION DEPTH: 12.6 m
REVIEWED BY: BW	COMPLETION DATE: May 6, 2024
Figure No. A1	Sheet 1 of 1

PROJECT: Boissevain Lagoon	DRILLER: Paddock Drilling Ltd.	TEST HOLE ID: **TH24-01A**
CLIENT: WSP Canada Inc.	DRILL RIG: Track Mounted Acker Mp5	PROJECT No: CA0017817.1877
LOCATION: Lift Station	DRILL METHOD: 125mm SSA/200mm HSA	ELEVATION: Not Surveyed

SAMPLE TYPE	■ Shelby Tube	▱ No Recovery	☒ SPT (N)	☰ Grab Sample	▥ Split-Pen	▮ Core
BACKFILL TYPE	■ Bentonite	⁘ Pea Gravel	▨ Drill Cuttings	▩ Grout	▥ Slough	⁙ Sand

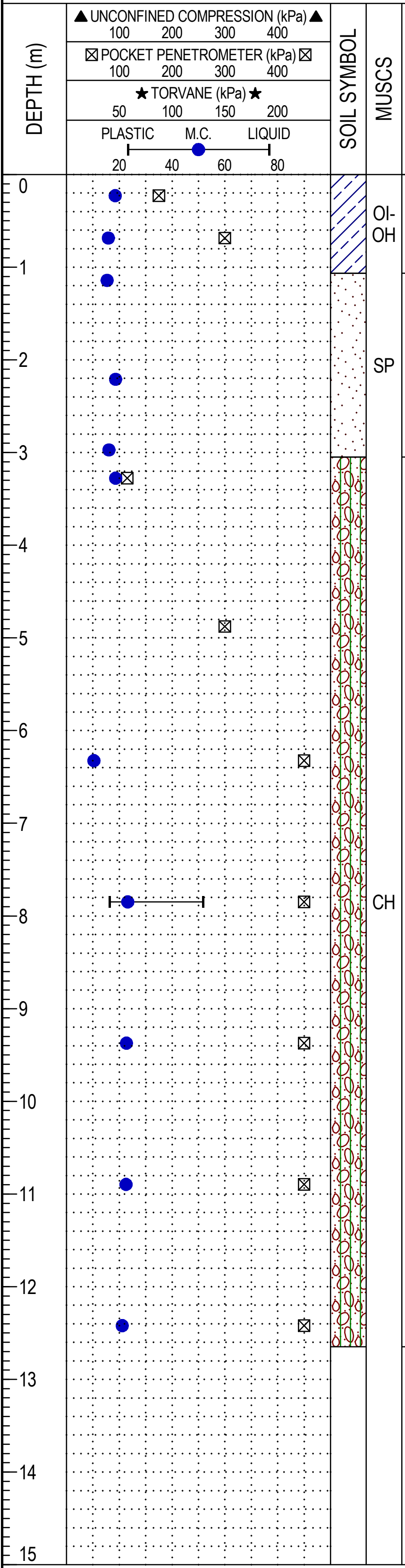
DEPTH (m)	▲ UNCONFINED COMPRESSION (kPa) ▲ 100 200 300 400 ☒ POCKET PENETROMETER (kPa) ☒ 100 200 300 400 ★ TORVANE (kPa) ★ 50 100 150 200 PLASTIC M.C. LIQUID 20 40 60 80	SOIL SYMBOL	SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NO	SPT (N)		COMMENTS	DEPTH (m)
0			SAME AS TH24-01 (Inferred)						
1									1
2									2
3								Water level measured at 2.5m below existing grade on 28 August 2024.	3
4									4
5									5
6									6
7									7
8									8
9									9
10									

WSP Canada Inc.	LOGGED BY: JC	COMPLETION DEPTH: 7.6 m
	REVIEWED BY: BW	COMPLETION DATE: May 7, 2024
	Figure No. A3	Sheet 1 of 1

CA0017817.1877 - BOISSEVAIN LAGOON.GPJ 24/09/13 10:36 AM (WPG - GEOTECH LOG 4)

PROJECT: Boissevain Lagoon	DRILLER: Paddock Drilling Ltd.	TEST HOLE ID: **TH24-02**
CLIENT: WSP Canada Inc.	DRILL RIG: Track Mounted Acker Mp5	PROJECT No: CA0017817.1877
LOCATION: Lift Station	DRILL METHOD: 125mm SSA/200mm HSA	ELEVATION: Not Surveyed

SAMPLE TYPE	▇ Shelby Tube	◸ No Recovery	☒ SPT (N)	☰ Grab Sample	▥ Split-Pen	▌ Core
BACKFILL TYPE	▇ Bentonite	⁘ Pea Gravel	▨ Drill Cuttings	⁘ Grout	▥ Slough	⁘ Sand



SOIL DESCRIPTION

ORGANIC CLAY - silty, trace sand, medium to high plastic, moist, stiff to very stiff, dark brown, occasional rootlets

- below 0.8m, sandy

SAND - trace silt, trace clay, poorly graded, fine grained, moist to very moist, loose (inferred), dark brown

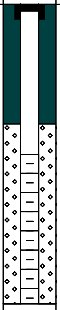
CLAY (TILL) - silty, trace sand, high plastic, moist, stiff, grey

- below 4.6m, very stiff

- below 6.1m, hard

TEST HOLE TERMINATED AT 12.6m BELOW EXISTING GRADE
Sloughing and seepage was observed between 1.1m and 3.0m during drilling.
Test hole remained open to 8.2m with water accumulated to 1.5m prior to installing monitoring well.

SAMPLE TYPE	SAMPLE NO	SPT (N)
☰	1	
☰	2	
☰	3	
☰	4	
☰	5	
☒	6	6
▇	7	
☒	8	41
☒	9	24
☒	10	36
☒	11	49
☒	12	49



COMMENTS

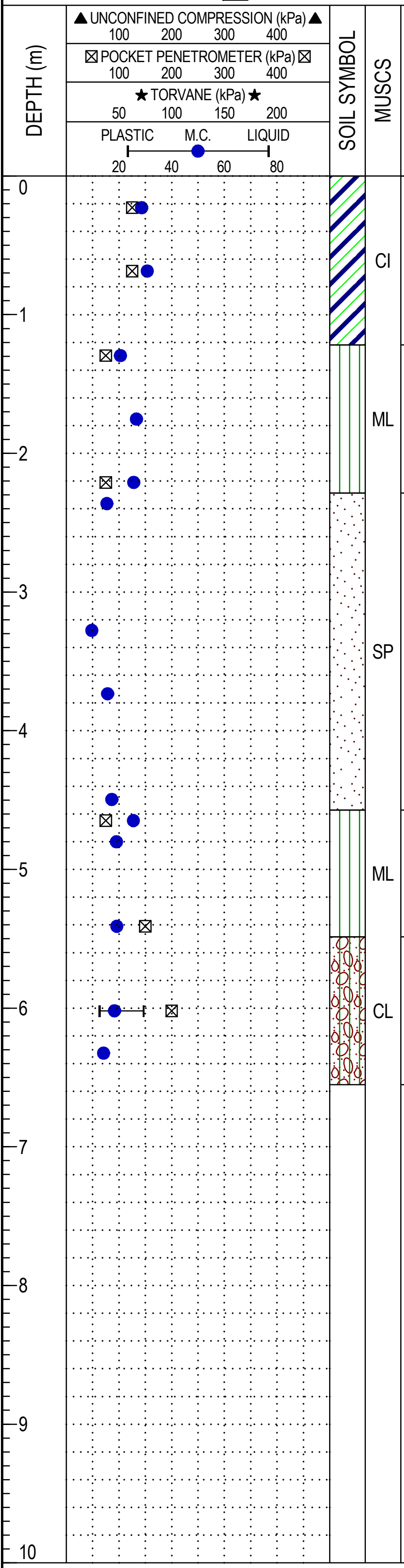
No water measured on 28 August 2024.

Particle Size Analysis - Sample 9 @ 7.6m:
Gravel= 0.0%
Sand= 7.2%
Silt= 56.2%
Clay= 36.6%

WSP Canada Inc.	LOGGED BY: JC	COMPLETION DEPTH: 12.6 m
	REVIEWED BY: BW	COMPLETION DATE: May 7, 2024
	Figure No. A2	Sheet 1 of 1

PROJECT: Boissevain Lagoon	DRILLER: Paddock Drilling Ltd.	TEST HOLE ID: **TH24-03**
CLIENT: WSP Canada Inc.	DRILL RIG: Track Mounted Mobile B-57	PROJECT No: CA0017817.1877
LOCATION: Lagoon Expansion	DRILL METHOD: 125mm SSA	ELEVATION: Not Surveyed

SAMPLE TYPE	▨ Shelby Tube	◺ No Recovery	☒ SPT (N)	☰ Grab Sample	▥ Split-Pen	▮ Core
BACKFILL TYPE	■ Bentonite	⁚ Pea Gravel	▨ Drill Cuttings	⁘ Grout	▥ Slough	⁘ Sand



SOIL DESCRIPTION

CLAY - silty to and silt, medium plastic, moist, stiff, dark brown to grey, occasional roots

- below 0.8m, no roots

SILT - some clay, low plastic, moist, firm, light grey

SAND - trace gravel, poorly graded, medium to coarse grained, very moist, compact (inferred), brown

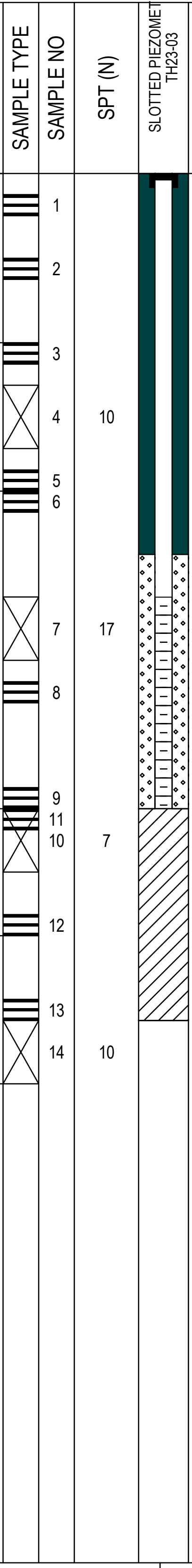
- below 3.4m, grey, compact

- below 3.7m, occasional till inclusions, moist

SILT - trace sand, trace to some clay, low plastic, moist, firm, dark grey

CLAY (TILL) - silty, trace sand, trace gravel, low plastic, stiff, dark grey

TEST HOLE TERMINATED AT 6.6m BELOW EXISTING GRADE
Sloughing and seepage was observed between 2.3m and 4.6m during drilling.
Test hole remained open to 3.0m with water accumulated to 0.3m prior to installation of monitoring well.



SAMPLE NO	SPT (N)
1	
2	
3	
4	10
5	
6	
7	17
8	
9	
11	
10	7
12	
13	
14	10

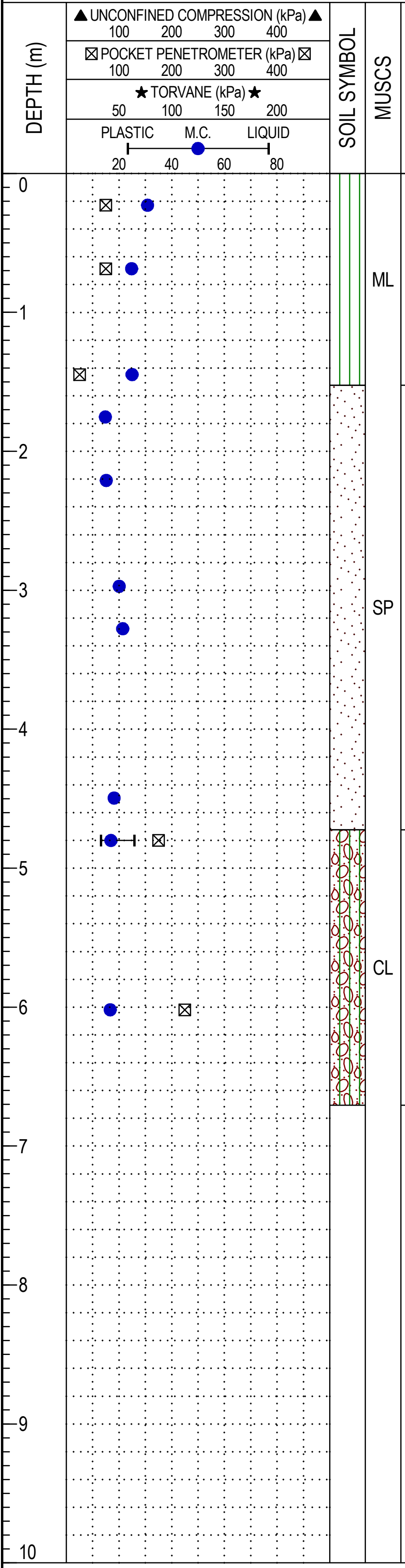
COMMENTS

Water level measured at 1.7m below existing grade on 28 August 2024.

WSP Canada Inc.	LOGGED BY: JC	COMPLETION DEPTH: 6.6 m
	REVIEWED BY: BW	COMPLETION DATE: May 8, 2024
	Figure No. A5	Sheet 1 of 1

PROJECT: Boissevain Lagoon	DRILLER: Paddock Drilling Ltd.	TEST HOLE ID: **TH24-04**
CLIENT: WSP Canada Inc.	DRILL RIG: Track Mounted Mobile B-57	PROJECT No: CA0017817.1877
LOCATION: Lagoon Expansion	DRILL METHOD: 125mm SSA	ELEVATION: Not Surveyed

SAMPLE TYPE	▩ Shelby Tube	▨ No Recovery	☒ SPT (N)	☰ Grab Sample	▥ Split-Pen	▮ Core
BACKFILL TYPE	■ Bentonite	⁚ Pea Gravel	▨ Drill Cuttings	⁛ Grout	▥ Slough	⁘ Sand



SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NO	SPT (N)	COMMENTS
SILT - some clay, low to medium plastic, moist, firm, brown	☰	1		
	☰	2		
- below 0.9m, light grey	☰	3		
SAND - trace gravel, poorly graded, fine grained, very moist, compact, brown	☒	4	20	
	☰	5		
	☰	6		
	☒	7	10	
- below 3.7m, grey	☰	8		
CLAY (TILL) - silty, trace sand, trace gravel, low plastic, moist, stiff, dark grey	☰	9	11	
	☒	10		
- below 6.1m, very stiff	☰	11		
	▩	12		

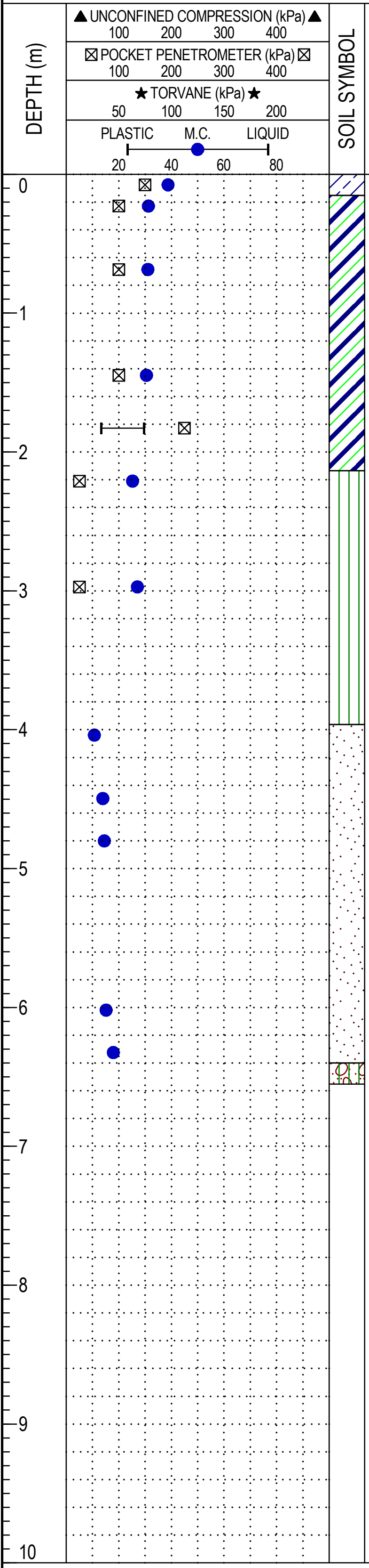
TEST HOLE TERMINATED AT 6.7m BELOW EXISTING GRADE
Sloughing and seepage was observed between 1.5m and 4.7m during drilling.
Test hole remained open to 1.5m with water accumulated to 0.6m prior to backfill.
Test hole backfilled with auger cuttings and bentonite.

WSP Canada Inc.	LOGGED BY: JC	COMPLETION DEPTH: 6.7 m
	REVIEWED BY: BW	COMPLETION DATE: May 8, 2024
	Figure No. A6	Sheet 1 of 1

PROJECT: Boissevain Lagoon	DRILLER: Paddock Drilling Ltd.	TEST HOLE ID: **TH24-05**
CLIENT: WSP Canada Inc.	DRILL RIG: Track Mounted Mobile B-57	PROJECT No: CA0017817.1877
LOCATION: Lagoon Expansion	DRILL METHOD: 125mm SSA	ELEVATION: Not Surveyed

SAMPLE TYPE: ■ Shelby Tube ▨ No Recovery ☒ SPT (N) ☰ Grab Sample ▥ Split-Pen ▮ Core

BACKFILL TYPE: ■ Bentonite ⁘ Pea Gravel ▨ Drill Cuttings ▩ Grout ▥ Slough ⁘ Sand



SOIL DESCRIPTION

OI — ORGANIC CLAY - silty, medium plastic, moist, stiff, dark brown, occasional grass roots

CI — CLAY - silty to and silt, medium plastic, moist, firm to stiff, grey

- below 1.4m, tan-brown

ML — SILT - trace clay, some sand, low plastic, moist, soft, brown

- below 3.7m, sandy

SP — SAND - some gravel, trace clay, trace silt, poorly graded, medium to coarse grained, very moist, compact, brown

- below 4.6m, grey

- below 5.9m, brown

CL — CLAY (TILL) - silty, trace sand, low plastic, moist, firm, dark grey

TEST HOLE TERMINATED AT 6.6m BELOW EXSITING GRADE
Sloughing was observed between 2.1m and 4.0m during drilling.
Seepage was observed between 4.0m and 6.4m during drilling.
Test hole remained open to 3.7m with water accumulated to 0.6m prior to backfill.
Test hole backfilled with auger cuttings and bentonite.

SAMPLE TYPE	SAMPLE NO	SPT (N)
☰	1	
☰	2	
☰	3	
☰	4	
■	5	
☰	6	
☰	7	
☒	8	2
☰	9	
☰	10	
☒	11	18
☰	12	
☒	13	6

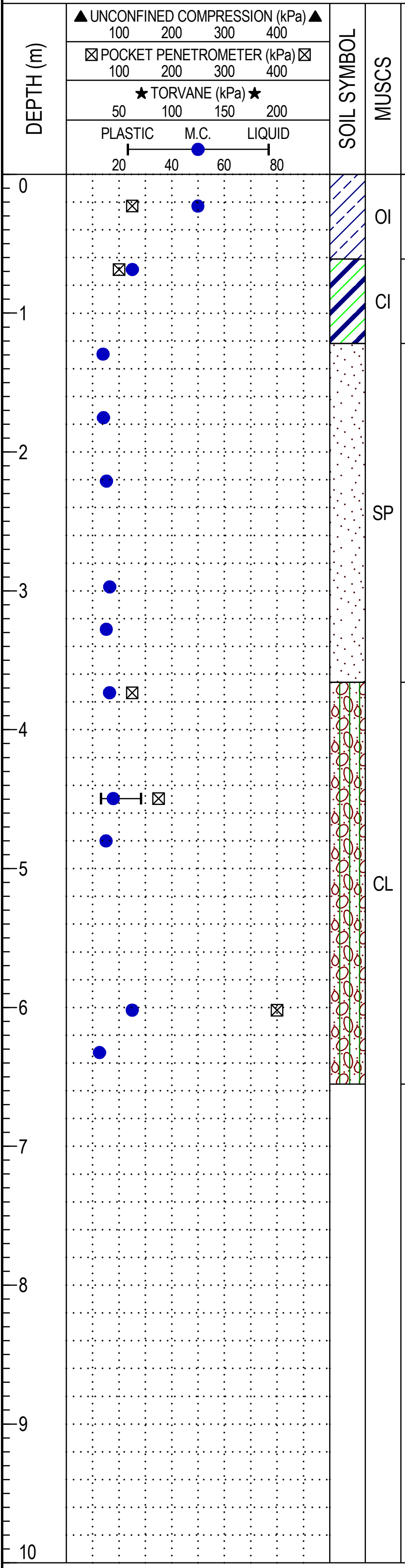
COMMENTS

Particle Size Analysis - Sample 5 @ 1.5m:
Gravel= 0.0%
Sand= 37.6%
Silt= 36.5%
Clay= 25.9%

WSP Canada Inc.	LOGGED BY: JC	COMPLETION DEPTH: 6.6 m
	REVIEWED BY: BW	COMPLETION DATE: May 8, 2024
	Figure No. A7	Sheet 1 of 1

PROJECT: Boissevain Lagoon	DRILLER: Paddock Drilling Ltd.	TEST HOLE ID: **TH24-06**
CLIENT: WSP Canada Inc.	DRILL RIG: Track Mounted Mobile B-57	PROJECT No: CA0017817.1877
LOCATION: Lagoon Expansion	DRILL METHOD: 125mm SSA	ELEVATION: Not Surveyed

SAMPLE TYPE	▒ Shelby Tube	▱ No Recovery	☒ SPT (N)	☰ Grab Sample	▥ Split-Pen	▮ Core
BACKFILL TYPE	■ Bentonite	⁚ Pea Gravel	▨ Drill Cuttings	⁘ Grout	▥ Slough	⁙ Sand



SOIL DESCRIPTION

OI – ORGANIC CLAY - silty, medium plastic, moist, stiff, dark brown, occasional grass roots,

CI – CLAY - silty, trace sand, medium plastic, moist, stiff, grey

- below 0.9m, some sand

SP – SAND - trace silt, trace clay, poorly graded, fine grained, very moist, loose (inferred), brown

- below 1.5m, trace gravel

- below 2.0m, compact

CL – CLAY (TILL) - silty, trace sand, trace gravel, low plastic, moist, stiff, dark grey

- below 4.6m, very stiff

TEST HOLE TERMINATED AT 6.6m BELOW EXISTING GRADE
Sloughing and seepage was observed between 1.2m and 3.7m during drilling.
Test hole remained open to 3.0m with water accumulated to 0.3m prior to backfill.
Test hole backfilled with auger cuttings and bentonite.

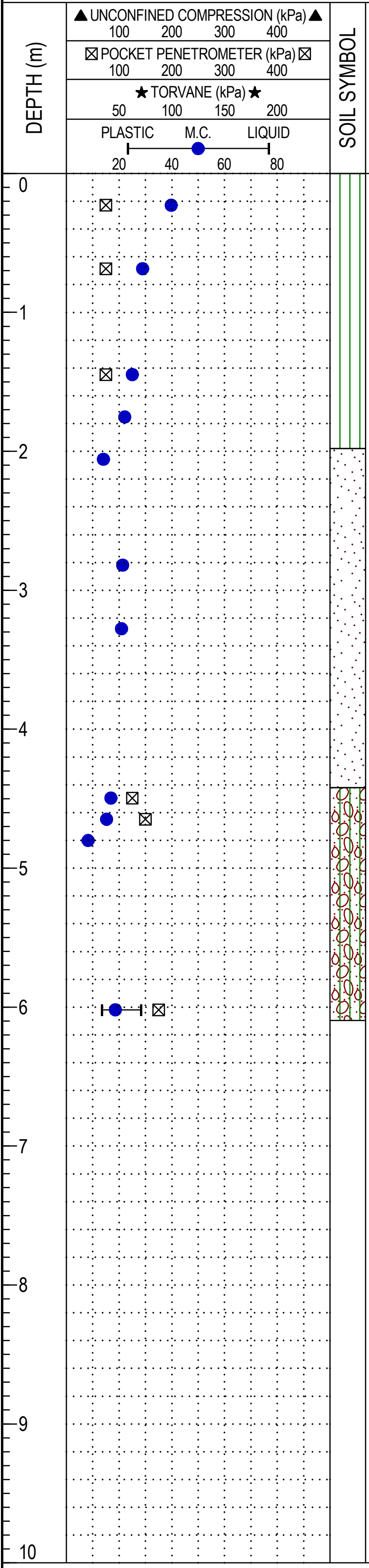
SAMPLE TYPE	SAMPLE NO	SPT (N)	COMMENTS
☰	1		
☰	2		
☰	3		
☒	4	24	
☰	5		
☰	6		
☒	7	13	
☰	8		
☰	9		
☒	10	26	Particle Size Analysis - Sample 9 @ 4.4m: Gravel= 0.0% Sand= 41.2% Silt= 37.1% Clay= 21.7%
☰	11		
☰	12	18	

WSP Canada Inc.

LOGGED BY: JC	COMPLETION DEPTH: 6.6 m
REVIEWED BY: BW	COMPLETION DATE: May 8, 2024
Figure No. A8	Sheet 1 of 1

PROJECT: Boissevain Lagoon	DRILLER: Paddock Drilling Ltd.	TEST HOLE ID: **TH24-07**
CLIENT: WSP Canada Inc.	DRILL RIG: Track Mounted Mobile B-57	PROJECT No: CA0017817.1877
LOCATION: Lagoon Expansion	DRILL METHOD: 125mm SSA	ELEVATION: Not Surveyed

SAMPLE TYPE	▩ Shelby Tube	▨ No Recovery	☒ SPT (N)	☰ Grab Sample	▥ Split-Pen	◧ Core
BACKFILL TYPE	■ Bentonite	⁙ Pea Gravel	▨ Drill Cuttings	⁙ Grout	▥ Slough	⁙ Sand



MUSCS	SOIL DESCRIPTION
ML	SILT - some clay, low plastic, moist, firm, dark brown, occasional roots - below 0.9m, no roots - below 1.8m, very stiff
SP	SAND - gravelly, poorly graded, medium to coarse grained, very moist, compact, brown - below 2.7m, fine grained, silty, greyish brown - below 3.0m, medium to coarse grained, trace silt, wet - below 3.8m, occasional till inclusions
CL	CLAY (TILL) - silty, trace sand, trace gravel, low plastic, moist, stiff, dark grey

SAMPLE TYPE	SAMPLE NO	SPT (N)
☰	1	
☰	2	
☰	3	
☒	4	22
☰	5	
☰	6	
☒	7	13
☰	8	
☰	10	
☒	9	7
☰	11	

TEST HOLE TERMINATED AT 6.1m BELOW EXISTING GRADE
Sloughing and seepage was observed between 2.0m and 4.4m during drilling.
Test hole remained open to 1.5m with water accumulated to 0.3m prior to backfill.
Test hole backfilled with auger cuttings and bentonite.

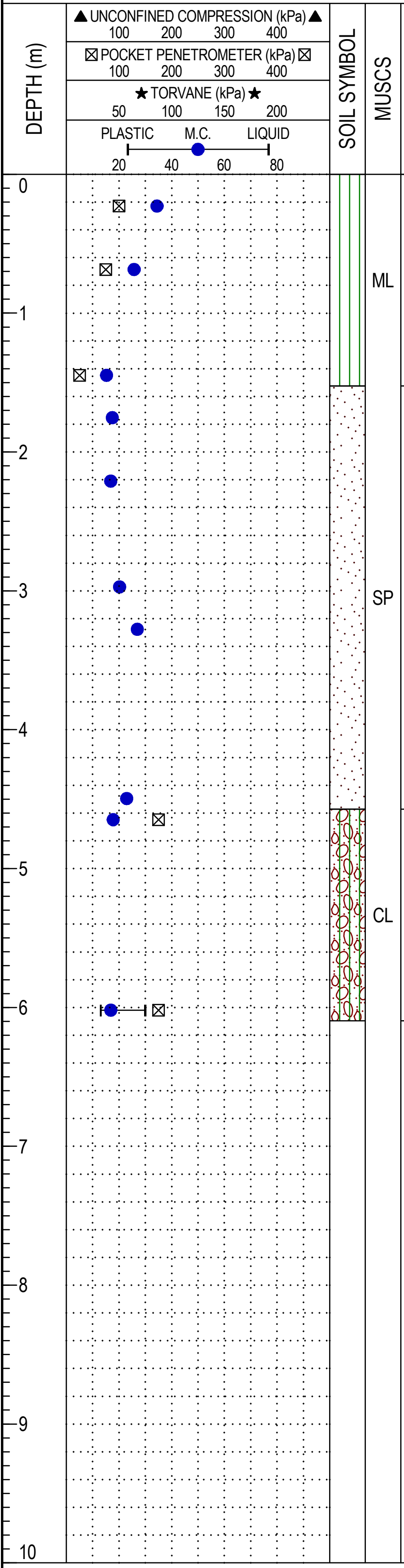
COMMENTS

Particle Size Analysis - Sample 11 @ 5.9m:
Gravel= 0.0%
Sand= 42.0%
Silt= 35.9%
Clay= 22.1%

WSP Canada Inc.	LOGGED BY: JC	COMPLETION DEPTH: 6.1 m
	REVIEWED BY: BW	COMPLETION DATE: May 8, 2024
	Figure No. A9	Sheet 1 of 1

PROJECT: Boissevain Lagoon	DRILLER: Paddock Drilling Ltd.	TEST HOLE ID: **TH24-08**
CLIENT: WSP Canada Inc.	DRILL RIG: Track Mounted Acker Mp5	PROJECT No: CA0017817.1877
LOCATION: Lagoon Expansion	DRILL METHOD: 125mm SSA/200mm HSA	ELEVATION: Not Surveyed

SAMPLE TYPE	▨ Shelby Tube	◿ No Recovery	☒ SPT (N)	☰ Grab Sample	▥ Split-Pen	▌ Core
BACKFILL TYPE	■ Bentonite	⁚ Pea Gravel	◿ Drill Cuttings	⁖ Grout	▥ Slough	⁘ Sand



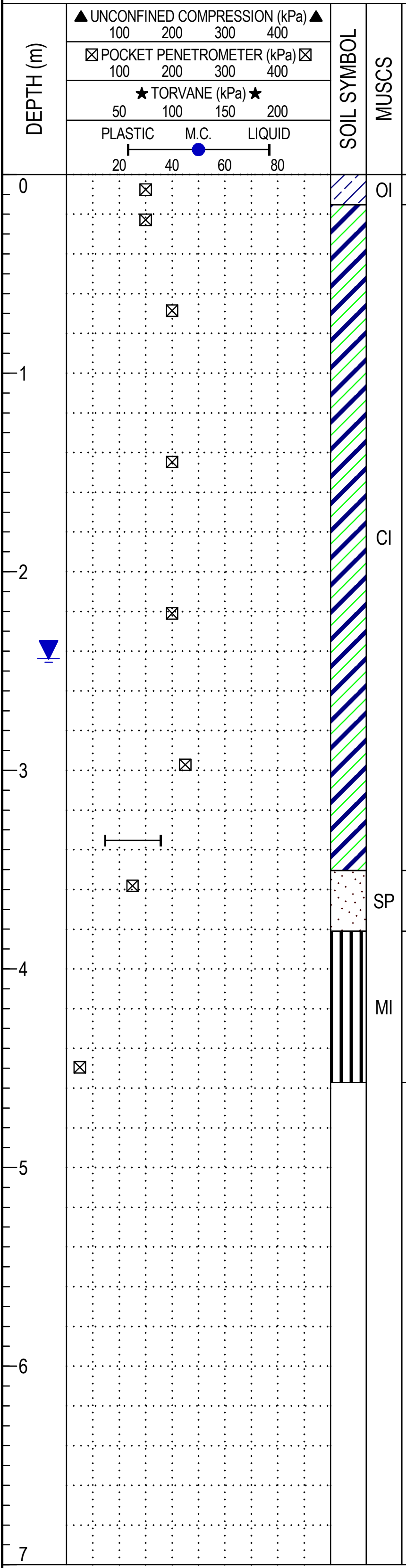
MUSCS	SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NO	SPT (N)	COMMENTS
ML	SILT - clayey, low to medium plastic, stiff, dark grey, occasional rootlets	☰	1		
	- below 0.6m, light grey	☰	2		
	- below 1.2m, some sand very moist	☰	3		
SP	SAND - trace to some silt, trace gravel, poorly graded, fine grained, wet, compact, brown	☒	4	20	
		☰	5		
		☰	6		
		☒	7	4	
	- below 3.5m, trace clay, grey, loose				
		☰	8		
CL	CLAY (TILL) - silty, trace sand, trace gravel, low plastic, moist, stiff, dark grey	☰	9		
		☰	10		
	TEST HOLE TERMINATED AT 6.1m BELOW EXISTING GRADE Sloughing and seepage was observed between 1.2m and 4.6m during drilling. Test hole remained open to 2.4m with water accumulated to 1.5m prior to backfill. Test hole backfilled with auger cuttings and bentonite.				Particle Size Analysis - Sample 10 @ 5.9m: Gravel= 0.0% Sand= 39.6% Silt= 38.2% Clay= 22.3%

WSP Canada Inc.	LOGGED BY: JC	COMPLETION DEPTH: 6.1 m
	REVIEWED BY: BW	COMPLETION DATE: May 7, 2024
	Figure No. A4	Sheet 1 of 1

PROJECT: Boissevain Borrow Source	DRILLER: Paddock Drilling Ltd.	TEST HOLE ID: **TH24-09**
CLIENT: Manitoba Water Services Board	DRILL RIG: Mobile B48	PROJECT No: CA0017817.1877
LOCATION: NW14-3-20W	DRILL METHOD: 125mm SSA	ELEVATION: Not Surveyed

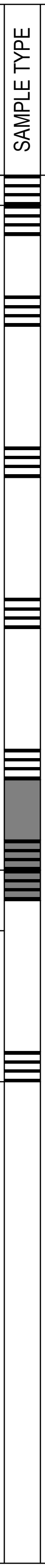
SAMPLE TYPE: ▨ Shelby Tube, ◿ No Recovery, ☒ SPT (N), ☰ Grab Sample, ▥ Split-Pen, ▌ Core

BACKFILL TYPE: ■ Bentonite, ⁚ Pea Gravel, ▨ Drill Cuttings, ⁘ Grout, ▥ Slough, ⁙ Sand



MUSCS	SOIL DESCRIPTION	SAMPLE NO	SPT (N)	COMMENTS
OI	ORGANIC CLAY - silty to and silt, trace gravel, trace sand, medium plastic, stiff, damp to moist, dark brown, frequent grass roots	1		
CI	CLAY - silty, trace sand, medium plastic, moist, stiff, brown - below 1.5m, trace gravel - below 2.3m, occasional to frequent silt laminations,stiff to very stiff	2 3 4 5 6 7 8		Particle Size Analysis - Sample 7 @ 3.0m: Gravel= 0.0% Sand= 26.8% Silt= 42.0% Clay= 31.2%
SP	SAND - trace gravel, poorly graded, fine grained, wet, loose (inferred), brown	9		
MI	SILT - clayey, medium plastic, moist, stiff, brown - below 4.0m, trace clay - below 4.3m, soft, black	10		

TEST HOLE TERMINATED AT 4.6m BELOW EXISTING GRADE
Sloughing and seepage was observed from 3.5m to 3.7m during drilling.
Test hole remained open to 3.7m with water level measured to 2.4m prior to backfilling.
Test hole backfilled with auger cuttings and bentonite.



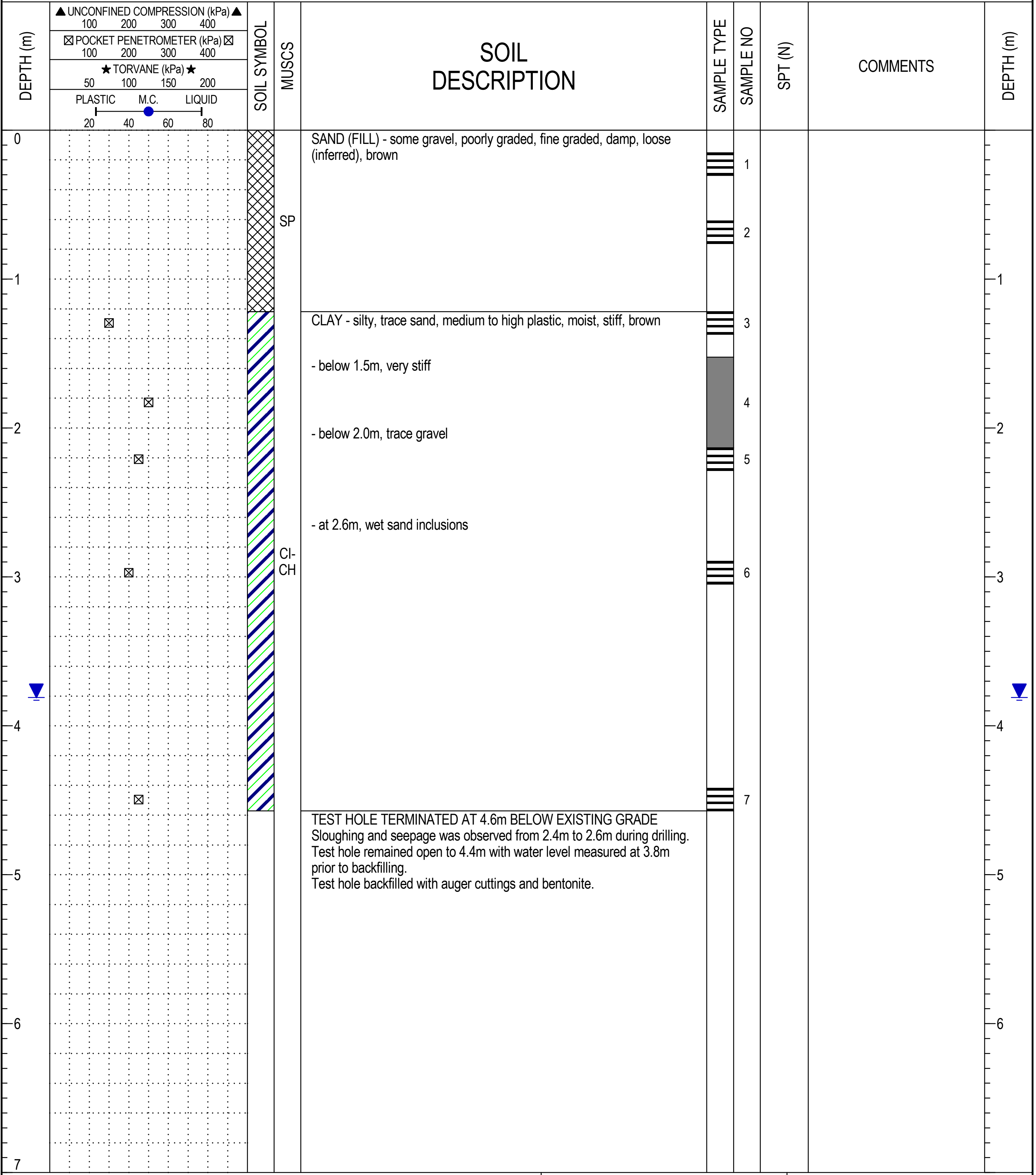
DEPTH (m): 1, 2, 3, 4, 5, 6

WSP Canada Inc.

LOGGED BY: JC	COMPLETION DEPTH: 4.6 m
REVIEWED BY: NB	COMPLETION DATE: August 1, 2024
Figure No. A1	Sheet 1 of 1

PROJECT: Boissevain Borrow Source	DRILLER: Paddock Drilling Ltd.	TEST HOLE ID: **TH24-10**
CLIENT: Manitoba Water Services Board	DRILL RIG: Mobile B48	PROJECT No: CA0017817.1877
LOCATION: NW14-3-20W	DRILL METHOD: 125mm SSA	ELEVATION: Not Surveyed

SAMPLE TYPE	▩ Shelby Tube	◿ No Recovery	☒ SPT (N)	☰ Grab Sample	▥ Split-Pen	▮ Core
BACKFILL TYPE	■ Bentonite	⁘ Pea Gravel	▨ Drill Cuttings	⁘ Grout	▥ Slough	⁘ Sand



SAND (FILL) - some gravel, poorly graded, fine graded, damp, loose (inferred), brown

CLAY - silty, trace sand, medium to high plastic, moist, stiff, brown

- below 1.5m, very stiff

- below 2.0m, trace gravel

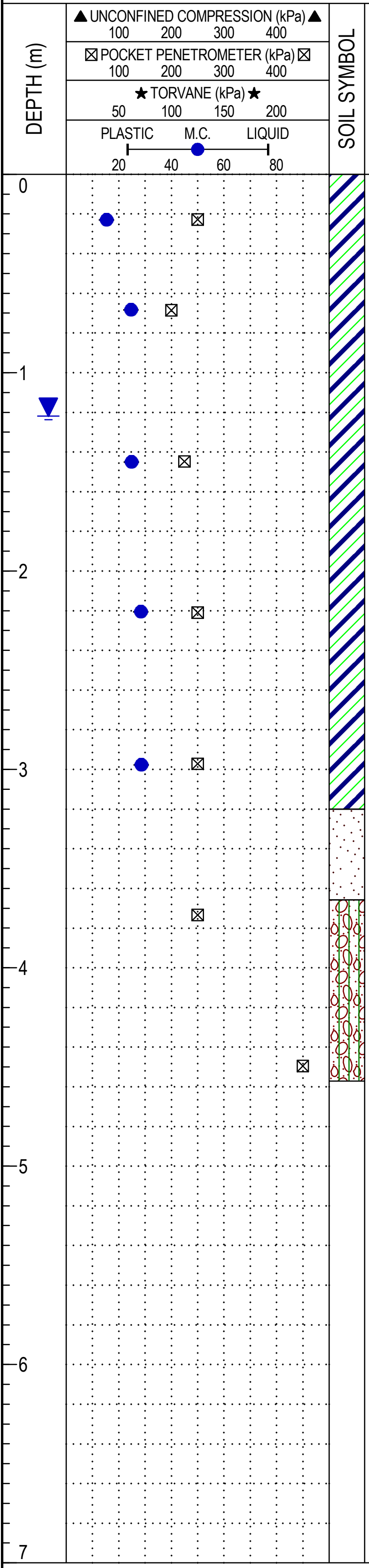
- at 2.6m, wet sand inclusions

TEST HOLE TERMINATED AT 4.6m BELOW EXISTING GRADE
Sloughing and seepage was observed from 2.4m to 2.6m during drilling.
Test hole remained open to 4.4m with water level measured at 3.8m prior to backfilling.
Test hole backfilled with auger cuttings and bentonite.

WSP Canada Inc.	LOGGED BY: JC	COMPLETION DEPTH: 4.6 m
	REVIEWED BY: NB	COMPLETION DATE: August 1, 2024
	Figure No. A2	Sheet 1 of 1

PROJECT: Boissevain Borrow Source	DRILLER: Paddock Drilling Ltd.	TEST HOLE ID: **TH24-11**
CLIENT: Manitoba Water Services Board	DRILL RIG: Mobile B48	PROJECT No: CA0017817.1877
LOCATION: NW14-3-20W	DRILL METHOD: 125mm SSA	ELEVATION: Not Surveyed

SAMPLE TYPE	▩ Shelby Tube	▱ No Recovery	☒ SPT (N)	☰ Grab Sample	▥ Split-Pen	◧ Core
BACKFILL TYPE	■ Bentonite	⁛ Pea Gravel	▨ Drill Cuttings	⁘ Grout	▥ Slough	⁙ Sand



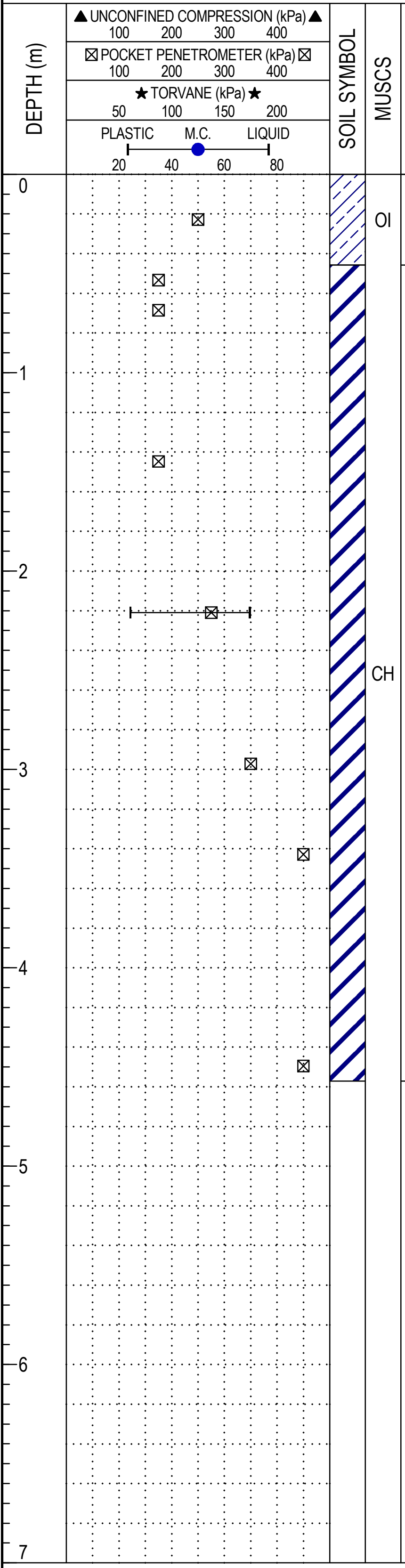
MUSCS	SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NO	SPT (N)	COMMENTS
CI	CLAY - silty, medium plastic, damp to moist, very stiff, dark brown, frequent grass roots to 0.2m - below 0.5m, brown - below 0.8m, occasional to frequent silt laminations - below 2.9m, trace sand	☰ ☰ ☰ ☰ ☰	1 2 3 4 5		
SP	SAND - trace gravel, poorly graded, fine grained, very moist to wet, loose (inferred), brown	☰	6		
CI	CLAY (TILL) - silty, trace sand, trace gravel, medium plastic, moist, very stiff, brown, frequent silt laminations - below 4.3m, grey, damp	☰ ☰	7 8		

TEST HOLE TERMINATED AT 4.6m BELOW EXISTING GRADE
Sloughing and seepage was observed between 3.2m to 3.7m during drilling.
Test hole remained open to 1.5m with water level measured at 1.2m prior to backfilling.
Test hole backfilled with auger cuttings and bentonite.

WSP Canada Inc.	LOGGED BY: JC	COMPLETION DEPTH: 4.6 m
	REVIEWED BY: NB	COMPLETION DATE: August 1, 2024
	Figure No. A3	Sheet 1 of 1

PROJECT: Boissevain Borrow Source	DRILLER: Paddock Drilling Ltd.	TEST HOLE ID: **TH24-12**
CLIENT: Manitoba Water Services Board	DRILL RIG: Mobile B48	PROJECT No: CA0017817.1877
LOCATION: NW14-3-20W	DRILL METHOD: 125mm SSA	ELEVATION: Not Surveyed

SAMPLE TYPE	■ Shelby Tube	▨ No Recovery	☒ SPT (N)	☰ Grab Sample	▥ Split-Pen	◧ Core
BACKFILL TYPE	■ Bentonite	⁘ Pea Gravel	▨ Drill Cuttings	⁘ Grout	▥ Slough	⁘ Sand



SOIL DESCRIPTION

ORGANIC CLAY - silty, medium plastic, damp to moist, very stiff, dark brown

CLAY - silty, high plastic, moist, stiff, greyish brown

- at 0.9m, sand inclusion
- below 1.1m, frequent silt laminations
- below 2.3m, very stiff
- below 3.4m, grey, hard

TEST HOLE TERMINATED AT 4.6m BELOW EXISTING GRADE
Caving was observed from surface to 1.5m during drilling.
No seepage was observed during drilling.
Test hole remained open to 3.4m with no accumulated water above the sloughed material prior to backfilling.
Test hole backfilled with auger cuttings and bentonite.

SAMPLE TYPE	SAMPLE NO	SPT (N)
☰	1	
☰	2	
☰	3	
☰	4	
☰	5	
☰	6	
☰	7	
☰	8	

COMMENTS

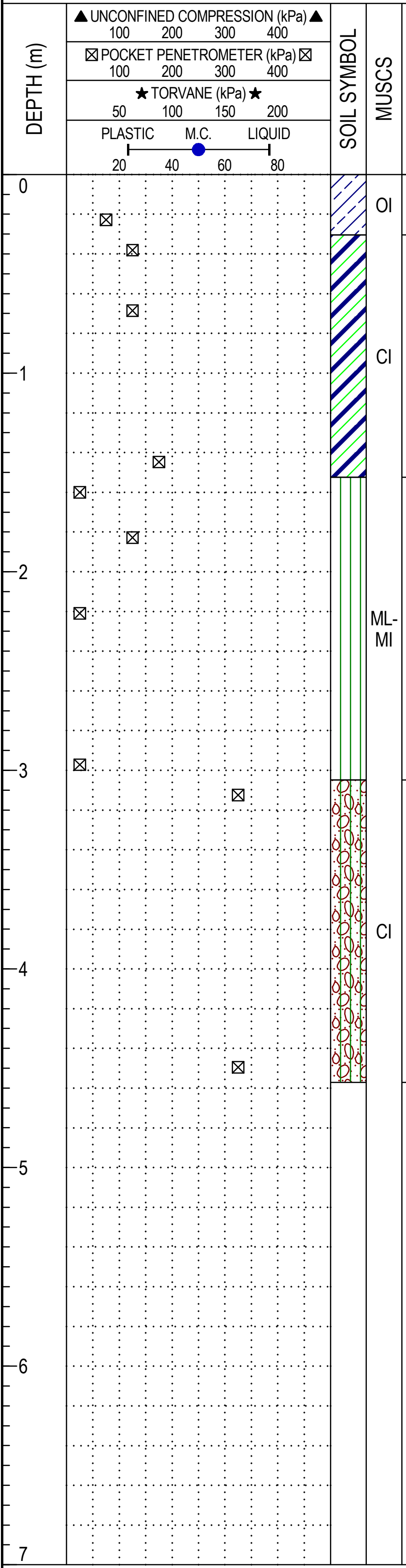
Particle Size Analysis - Sample 5 @ 2.1m:
Gravel= 0.0%
Sand= 1.3%
Silt= 51.8%
Clay= 46.9%

WSP Canada Inc.	LOGGED BY: JC	COMPLETION DEPTH: 4.6 m
	REVIEWED BY: NB	COMPLETION DATE: August 1, 2024
	Figure No. A4	Sheet 1 of 1

PROJECT: Boissevain Borrow Source	DRILLER: Paddock Drilling Ltd.	TEST HOLE ID: **TH24-13**
CLIENT: Manitoba Water Services Board	DRILL RIG: Mobile B48	PROJECT No: CA0017817.1877
LOCATION: NW14-3-20W	DRILL METHOD: 125mm SSA	ELEVATION: Not Surveyed

SAMPLE TYPE: ▨ Shelby Tube ◪ No Recovery ☒ SPT (N) ☰ Grab Sample ▥ Split-Pen ◨ Core

BACKFILL TYPE: ■ Bentonite ⬚ Pea Gravel ▧ Drill Cuttings ⬚ Grout ▥ Slough ⬚ Sand



MUSCS	SOIL DESCRIPTION	SAMPLE NO	SPT (N)	COMMENTS
OI	ORGANIC CLAY - and silt, medium plastic, moist, firm, dark brown	1		
CI	CLAY - and silt, medium plastic, moist, stiff, grey - below 0.9m, trace sand, brown	2 3 4		
ML-MI	SILT - some clay, low to medium plastic, moist to very moist, soft, brown, occasional clay inclusions	6 5 7 8		
CI	CLAY (TILL) - silty, medium plastic, moist, very stiff, grey	9 10		

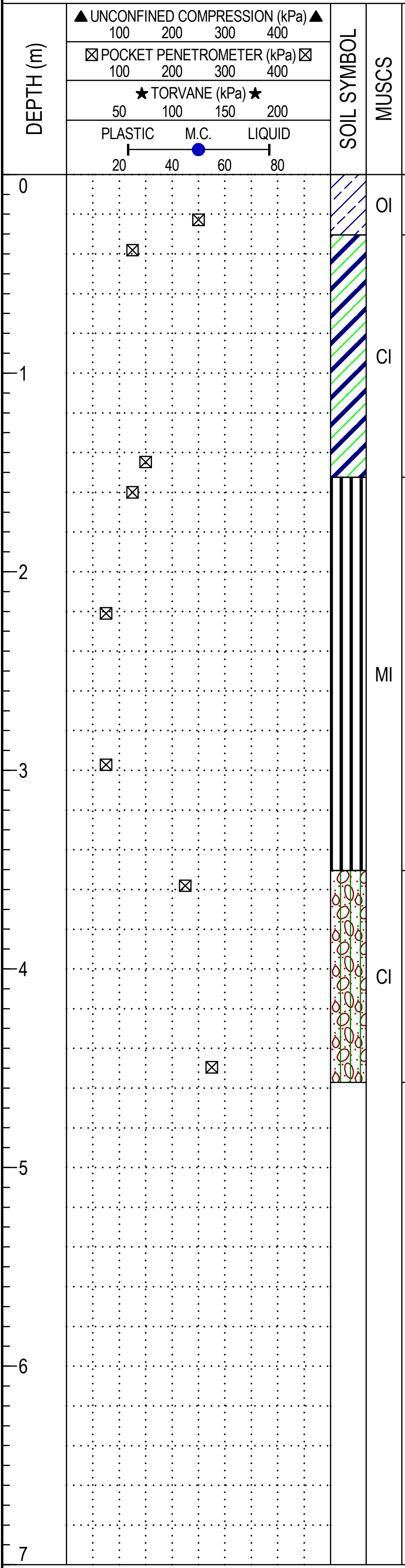
TEST HOLE TERMINATED AT 4.6m BELOW EXISTING GRADE
Sloughing and seepage was observed from 1.5m to 3.0m during drilling.
Test hole remained open to 2.1m with no water accumulated above the sloughed material prior to backfilling.
Test hole backfilled with auger cuttings and bentonite.

WSP Canada Inc.

LOGGED BY: JC	COMPLETION DEPTH: 4.6 m
REVIEWED BY: NB	COMPLETION DATE: August 1, 2024
Figure No. A5	Sheet 1 of 1

PROJECT: Boissevain Borrow Source	DRILLER: Paddock Drilling Ltd.	TEST HOLE ID: **TH24-15**
CLIENT: Manitoba Water Services Board	DRILL RIG: Mobile B48	PROJECT No: CA0017817.1877
LOCATION: NW14-3-20W	DRILL METHOD: 125mm SSA	ELEVATION: Not Surveyed

SAMPLE TYPE	■ Shelby Tube	▨ No Recovery	☒ SPT (N)	☰ Grab Sample	▥ Split-Pen	▮ Core
BACKFILL TYPE	■ Bentonite	⁘ Pea Gravel	▨ Drill Cuttings	⁘ Grout	▥ Slough	⁘ Sand



SOIL DESCRIPTION

ORGANIC CLAY - silty, medium plastic, moist, very stiff, dark brown

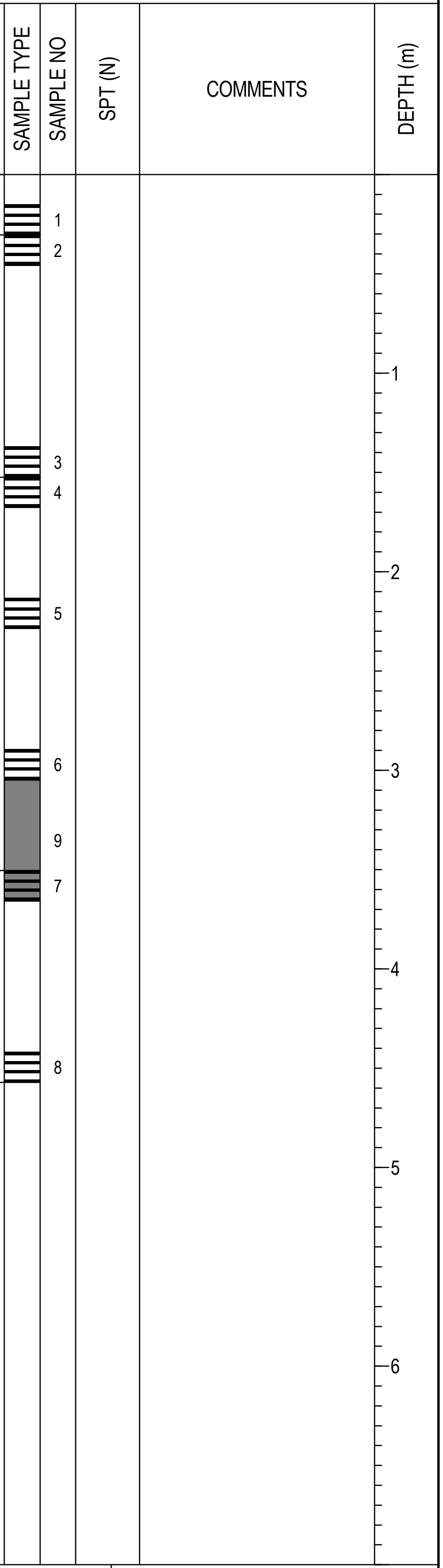
CLAY - silty to and silt, trace to some sand, medium plastic, moist, stiff, brown

SILT - clayey, medium plastic, moist, stiff, brown

- below 2.3m, firm, trace to some clay

CLAY (TILL) - silty, trace sand, trace gravel, medium plastic, moist, very stiff, brown

TEST HOLE TERMINATED AT 4.6m BELOW EXISTING GRADE
Sloughing was observed from 2.3m to 3.4m during drilling.
No seepage was observed during drilling.
Test hole remained open to 4.3m with no water accumulated above the sloughed material prior to backfilling.
Test hole backfilled with auger cuttings and bentonite.

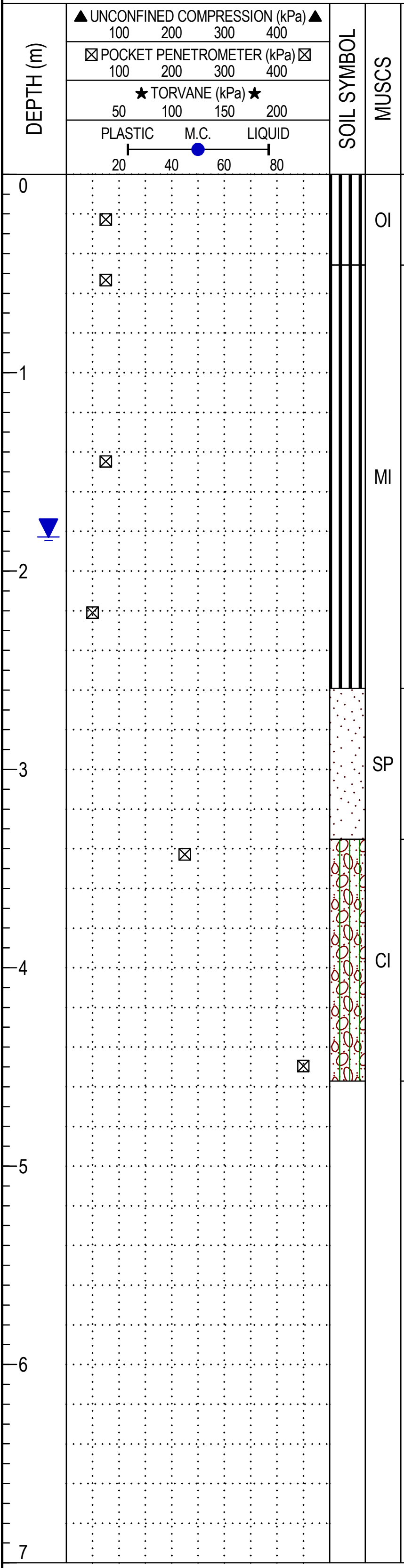


WSP Canada Inc.

LOGGED BY: JC	COMPLETION DEPTH: 4.6 m
REVIEWED BY: NB	COMPLETION DATE: August 1, 2024
Figure No. A6	Sheet 1 of 1

PROJECT: Boissevain Borrow Source	DRILLER: Paddock Drilling Ltd.	TEST HOLE ID: **TH24-16**
CLIENT: Manitoba Water Services Board	DRILL RIG: Mobile B48	PROJECT No: CA0017817.1877
LOCATION: NW14-3-20W	DRILL METHOD: 125mm SSA	ELEVATION: Not Surveyed

SAMPLE TYPE	▨ Shelby Tube	◿ No Recovery	☒ SPT (N)	☰ Grab Sample	▥ Split-Pen	◧ Core
BACKFILL TYPE	■ Bentonite	⁘ Pea Gravel	◿ Drill Cuttings	⁘ Grout	▥ Slough	⁘ Sand



SOIL DESCRIPTION

ORGANIC SILT - some clay, medium plastic, moist, firm, dark brown

SILT - some clay, medium plastic, moist, firm, light grey

- below 0.8m, tan-brown

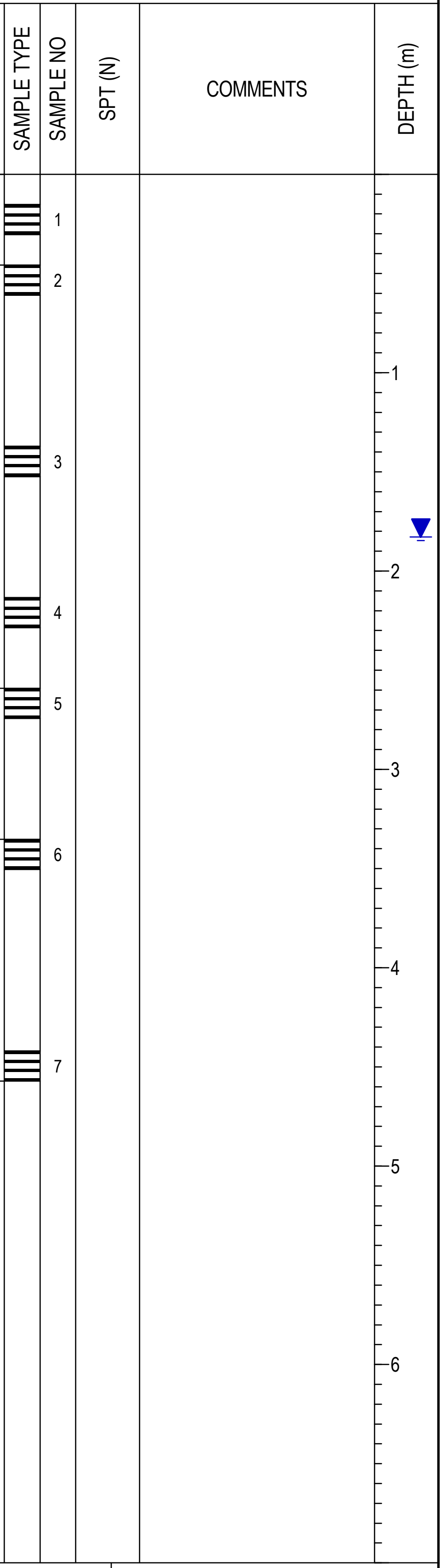
SAND - trace silt, poorly graded, fine grained, very moist, loose (inferred), brown

CLAY (TILL) - silty, trace sand, trace gravel, medium plastic, moist, very stiff, brown

- below 4.0m, grey

- below 4.4m, hard

TEST HOLE TERMINATED AT 4.6m BELOW EXISTING GRADE
Sloughing and seepage was observed from 2.6m to 3.4m during drilling.
Test hole remained open to 4.0m with water level measured to 1.8m prior to backfilling.
Test hole backfilled with auger cuttings and bentonite.



LOGGED BY: JC	COMPLETION DEPTH: 4.6 m
REVIEWED BY: NB	COMPLETION DATE: August 1, 2024
Figure No. A7	Sheet 1 of 1

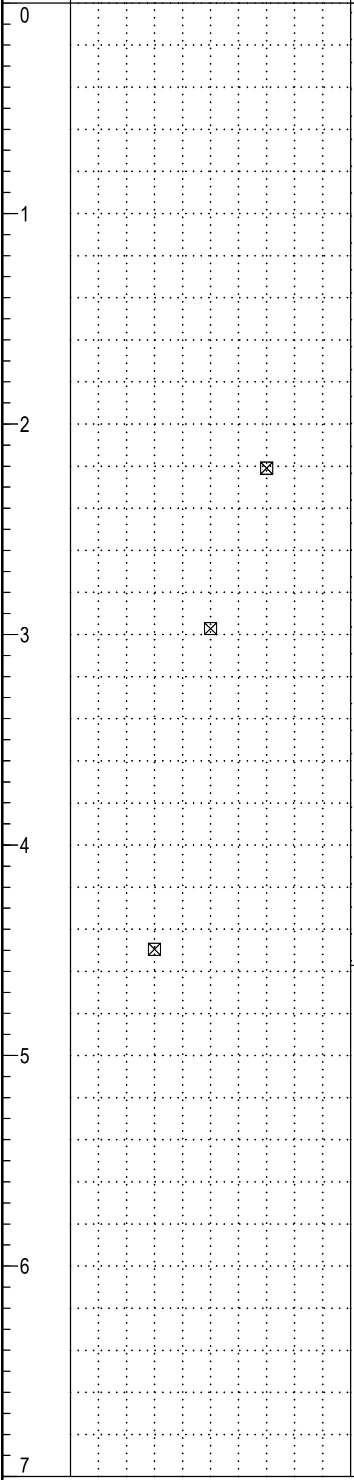
WSP Canada Inc.

PROJECT: Boissevain Borrow Source	DRILLER: Paddock Drilling Ltd.	TEST HOLE ID: **TH24-18**
CLIENT: Manitoba Water Services Board	DRILL RIG: Mobile B48	PROJECT No: CA0017817.1877
LOCATION: NW13-3-20W	DRILL METHOD: 125mm SSA	ELEVATION: Not Surveyed

SAMPLE TYPE: ▨ Shelby Tube ◪ No Recovery ☒ SPT (N) ☰ Grab Sample ▥ Split-Pen ◧ Core

BACKFILL TYPE: ■ Bentonite ⁚ Pea Gravel ◪ Drill Cuttings ⁘ Grout ▥ Slough ⁙ Sand

▲ UNCONFINED COMPRESSION (kPa) ▲ 100 200 300 400
☒ POCKET PENETROMETER (kPa) ☒ 100 200 300 400
★ TORVANE (kPa) ★ 50 100 150 200
PLASTIC M.C. LIQUID 20 40 60 80



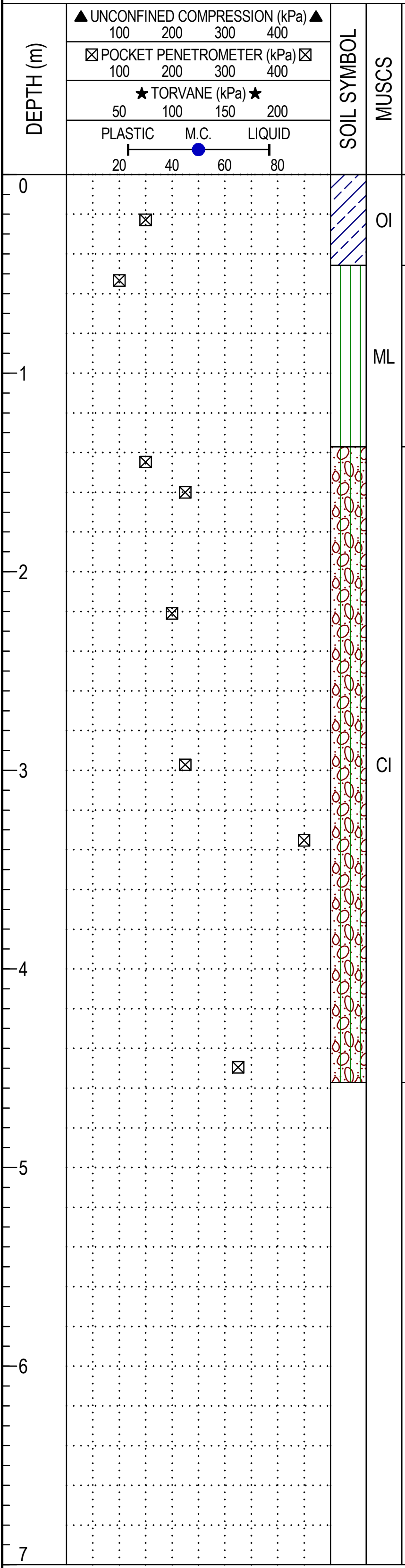
DEPTH (m)	SOIL SYMBOL	MUSCS	SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NO	SPT (N)	COMMENTS
0 – 4.6	⁚	SP	SAND - trace silt, poorly graded, fine grained, damp to moist, loose (inferred), dark brown, grass roots to 0.2m - below 0.2m, brown - below 1.5m, moist - below 2.1m, occasional clay inclusions - below 3.0m, trace to some clay	☰ ☰ ☰ ☰ ☰ ☰	1 2 3 4 5 6		

TEST HOLE TERMINATED AT 4.6m BELOW EXISTING GRADE
No sloughing or seepage was observed during drilling.
Test hole remained open to 4.6m with no water accumulated prior to backfilling.
Test hole backfilled with auger cuttings and bentonite.

WSP Canada Inc.	LOGGED BY: JC	COMPLETION DEPTH: 4.6 m
	REVIEWED BY: NB	COMPLETION DATE: August 2, 2024
	Figure No. A8	Sheet 1 of 1

PROJECT: Boissevain Borrow Source	DRILLER: Paddock Drilling Ltd.	TEST HOLE ID: **TH24-19**
CLIENT: Manitoba Water Services Board	DRILL RIG: Mobile B48	PROJECT No: CA0017817.1877
LOCATION: NW13-3-20W	DRILL METHOD: 125mm SSA	ELEVATION: Not Surveyed

SAMPLE TYPE	▒ Shelby Tube	▱ No Recovery	☒ SPT (N)	☰ Grab Sample	▥ Split-Pen	◧ Core
BACKFILL TYPE	■ Bentonite	⁚ Pea Gravel	▨ Drill Cuttings	⁘ Grout	▥ Slough	⁙ Sand



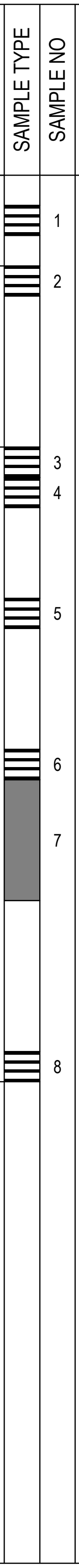
SOIL DESCRIPTION

ORGANIC CLAY - silty, trace sand, medium plastic, moist, stiff, dark brown, occasional rootlets

SILT - some clay, sandy, low plastic, moist, firm to stiff, grey

CLAY (TILL) - silty, trace sand, trace gravel, medium plastic, moist, stiff, brown
- below 1.5m, very stiff
- below 2.7m, grey
- below 3.0m, no sand or gravel

TEST HOLE TERMINATED AT 4.6m BELOW EXISTING GRADE
No sloughing or seepage was observed during drilling.
Test hole remained open to 4.6m with no accumulated water prior to backfilling.
Test hole backfilled with auger cuttings and bentonite.



SPT (N)

COMMENTS

DEPTH (m)

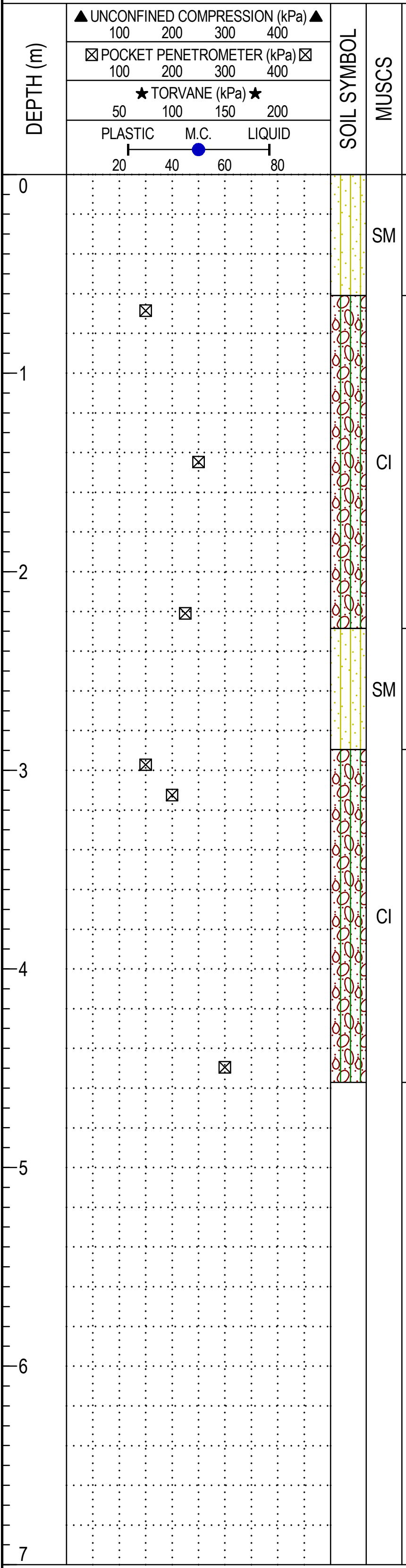
WSP Canada Inc.

LOGGED BY: JC	COMPLETION DEPTH: 4.6 m
REVIEWED BY: NB	COMPLETION DATE: August 2, 2024
Figure No. A9	Sheet 1 of 1

PROJECT: Boissevain Borrow Source	DRILLER: Paddock Drilling Ltd.	TEST HOLE ID: **TH24-20**
CLIENT: Manitoba Water Services Board	DRILL RIG: Mobile B48	PROJECT No: CA0017817.1877
LOCATION: NW13-3-20W	DRILL METHOD: 125mm SSA	ELEVATION: Not Surveyed

SAMPLE TYPE: ■ Shelby Tube, ◿ No Recovery, ☒ SPT (N), ☰ Grab Sample, ▥ Split-Pen, ◧ Core

BACKFILL TYPE: ■ Bentonite, ⁘ Pea Gravel, ▨ Drill Cuttings, ⁘ Grout, ▥ Slough, ⁘ Sand



SOIL DESCRIPTION	SAMPLE NO	SPT (N)	COMMENTS
SAND - trace to some clay, trace silt, poorly graded, fine grained, moist, compact (inferred), brown	1		
CLAY(TILL) - silty, trace sand, trace gravel, medium plastic, moist, stiff, brown	2 3 4		
SAND - silty, trace to some clay, poorly graded, fine grained, moist, compact (inferred), brown - below 2.7m, some clay	5		
CLAY(TILL) - sandy, some silt, trace gravel, medium plastic, moist, very stiff, brown - below 4.3m, occasional to frequent silt laminations, trace sand	6 7 8		

TEST HOLE TERMINATED AT 4.6m BELOW EXISTING GRADE
No sloughing or seepage was observed during drilling.
Test hole remained open to 4.6m with no accumulated water prior to backfilling.
Test hole backfilled with auger cuttings and bentonite.

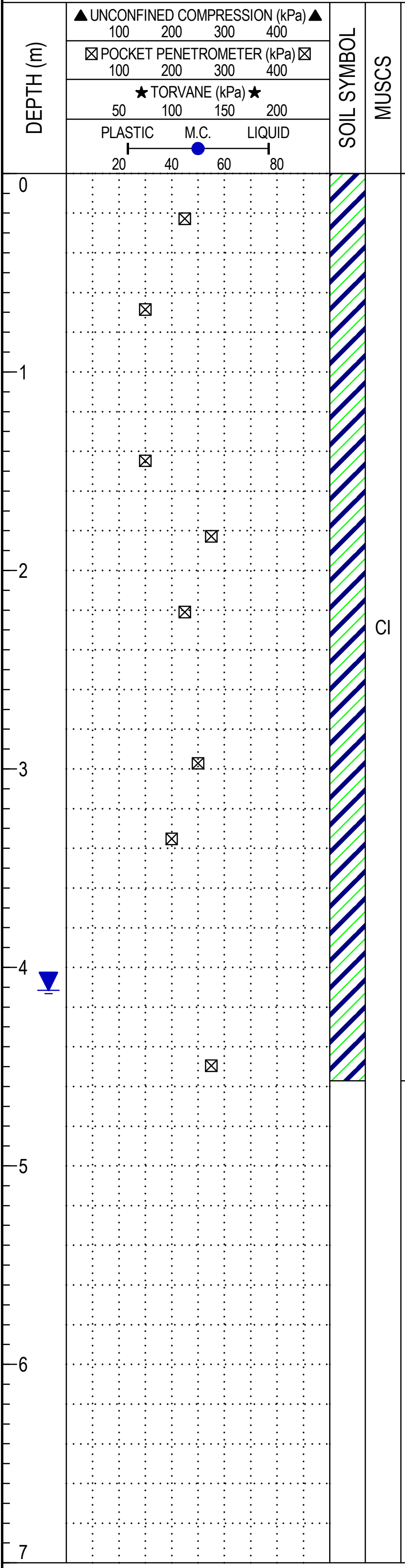
DEPTH (m): 1, 2, 3, 4, 5, 6

WSP Canada Inc.	LOGGED BY: JC	COMPLETION DEPTH: 4.6 m
	REVIEWED BY: NB	COMPLETION DATE: August 2, 2024
	Figure No. A10	Sheet 1 of 1

PROJECT: Boissevain Borrow Source	DRILLER: Paddock Drilling Ltd.	TEST HOLE ID: **TH24-21**
CLIENT: Manitoba Water Services Board	DRILL RIG: Mobile B48	PROJECT No: CA0017817.1877
LOCATION: NW13-3-20W	DRILL METHOD: 125mm SSA	ELEVATION: Not Surveyed

SAMPLE TYPE: ▇ Shelby Tube ◺ No Recovery ☒ SPT (N) ☰ Grab Sample ▥ Split-Pen ◧ Core

BACKFILL TYPE: ▇ Bentonite ⁚ Pea Gravel ▨ Drill Cuttings ⁙ Grout ▥ Slough ⁙ Sand



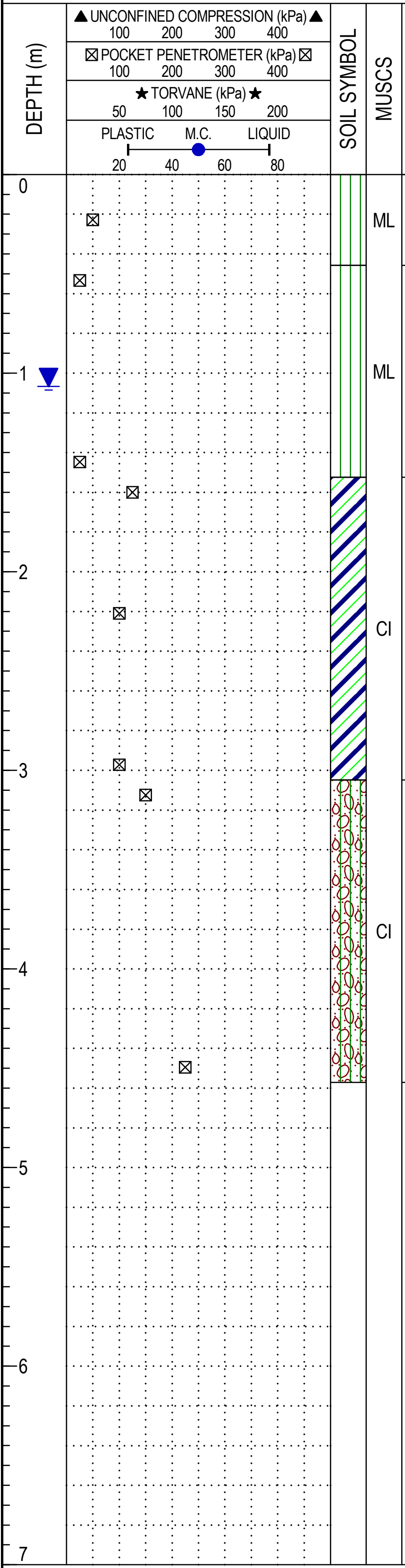
MUSCS	SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NO	SPT (N)	COMMENTS
CI	CLAY(TILL) - silty, trace sand, trace gravel, medium plastic, moist, very stiff, brown, occasional rootlets to 0.3m	☰	1		
		☰	2		
		☰	3		
		▇	4		
		☰	5		
		☰	6		
		▇	7		
		☰	8		

TEST HOLE TERMINATED AT 4.6m BELOW EXISTING GRADE
Sloughing and seepage was observed from 3.0m and 3.4m during drilling.
Test hole remained open to 4.3m with water level measured to 4.1m prior to backfilling.
Test hole backfilled with auger cuttings and bentonite.

WSP Canada Inc.	LOGGED BY: JC	COMPLETION DEPTH: 4.6 m
	REVIEWED BY: NB	COMPLETION DATE: August 2, 2024
	Figure No. A11	Sheet 1 of 1

PROJECT: Boissevain Borrow Source	DRILLER: Paddock Drilling Ltd.	TEST HOLE ID: **TH24-22**
CLIENT: Manitoba Water Services Board	DRILL RIG: Mobile B48	PROJECT No: CA0017817.1877
LOCATION: NW13-3-20W	DRILL METHOD: 125mm SSA	ELEVATION: Not Surveyed

SAMPLE TYPE	▩ Shelby Tube	▨ No Recovery	☒ SPT (N)	☰ Grab Sample	▥ Split-Pen	◧ Core
BACKFILL TYPE	■ Bentonite	⁙ Pea Gravel	▨ Drill Cuttings	⁙ Grout	▥ Slough	⁙ Sand



SOIL DESCRIPTION

ORGANIC SILT - trace clay, trace sand, low plastic, soft, dark brown, occasional roots

SILT - trace clay, some sand, low plastic, moist to very moist, soft, grey

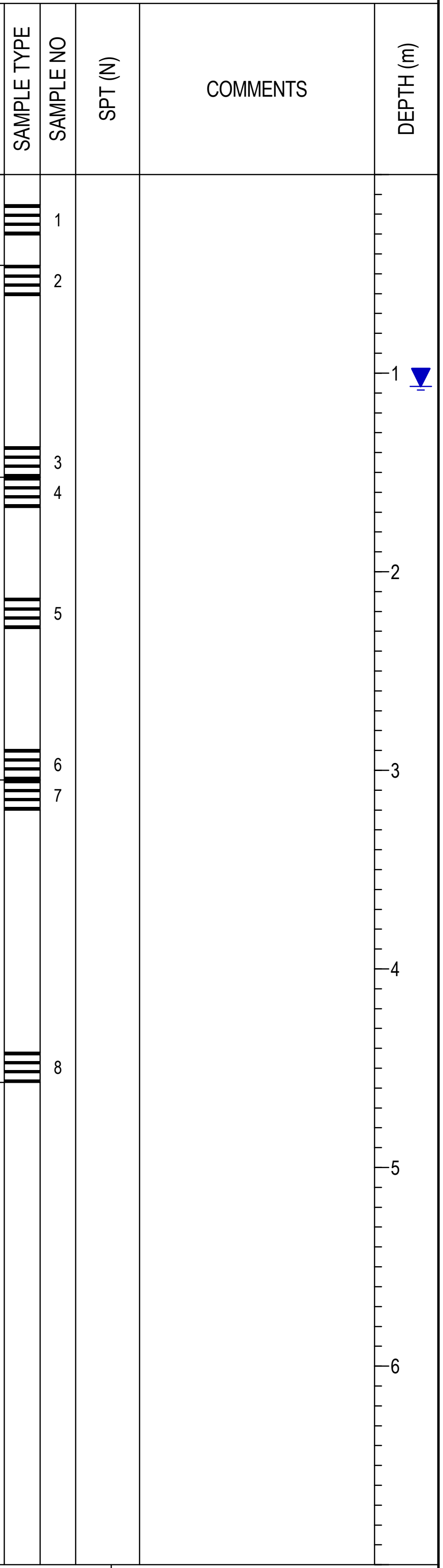
- below 1.2m, mottled grey/brown

CLAY - and silt, trace sand, medium plastic, moist, stiff, brown

CLAY (TILL) - silty, trace sand, trace gravel, medium plastic, moist, stiff, grey

- below 4.4m, very stiff

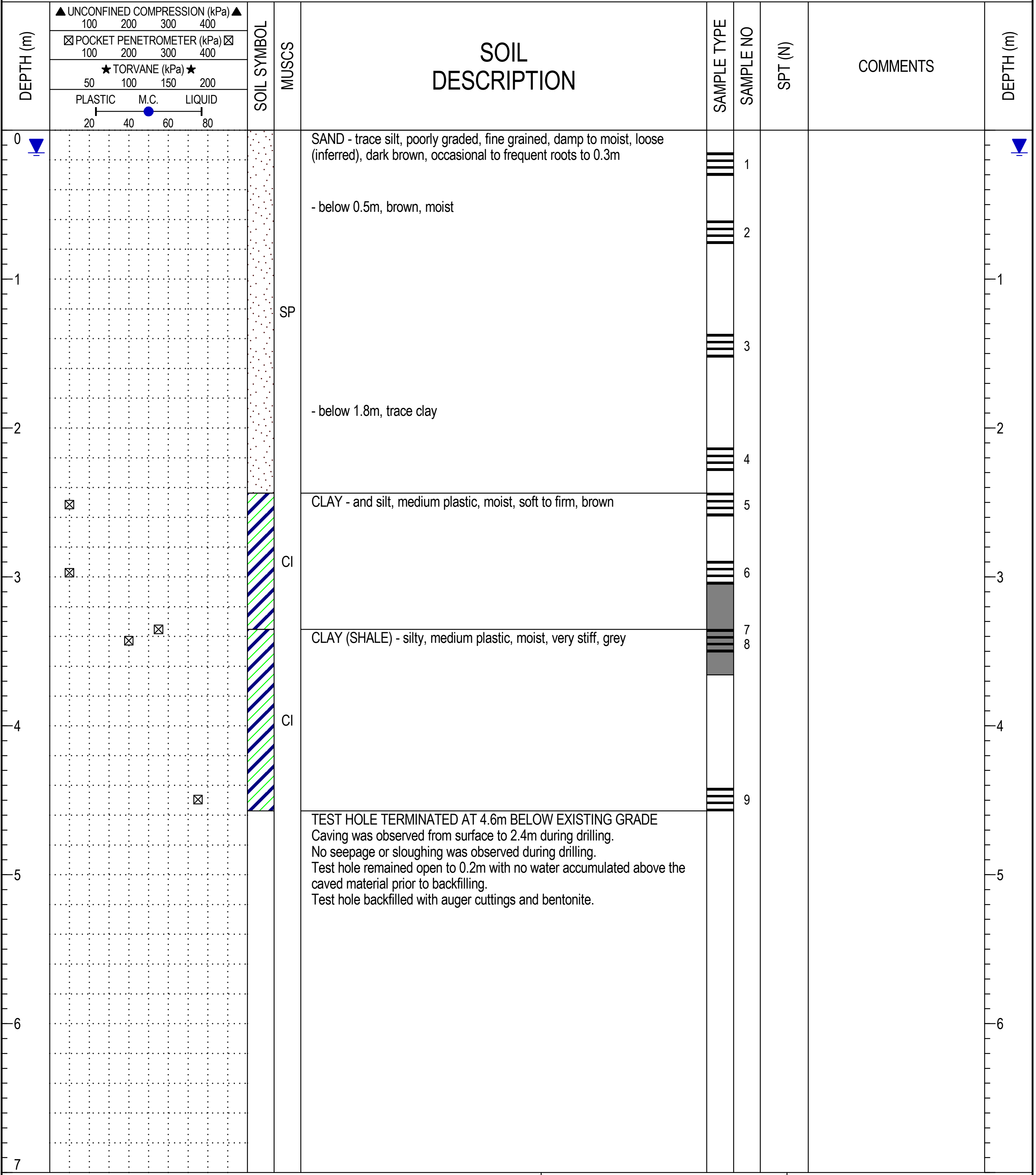
TEST HOLE TERMINATED AT 4.6m BELOW EXISTING GRADE
Sloughing and seepage was observed from 0.5m to 1.5m during drilling.
Test hole remained open to 1.2m with water level measured to 1.1m prior to backfilling.
Test hole backfilled with auger cuttings and bentonite.



WSP Canada Inc.	LOGGED BY: JC	COMPLETION DEPTH: 4.6 m
	REVIEWED BY: NB	COMPLETION DATE: August 2, 2024
	Figure No. A12	Sheet 1 of 1

PROJECT: Boissevain Borrow Source	DRILLER: Paddock Drilling Ltd.	TEST HOLE ID: **TH24-23**
CLIENT: Manitoba Water Services Board	DRILL RIG: Mobile B48	PROJECT No: CA0017817.1877
LOCATION: NW13-3-20W	DRILL METHOD: 125mm SSA	ELEVATION: Not Surveyed

SAMPLE TYPE	■ Shelby Tube	▨ No Recovery	☒ SPT (N)	☰ Grab Sample	▥ Split-Pen	◧ Core
BACKFILL TYPE	■ Bentonite	⁘ Pea Gravel	▨ Drill Cuttings	⁘ Grout	▥ Slough	⁘ Sand



SAND - trace silt, poorly graded, fine grained, damp to moist, loose (inferred), dark brown, occasional to frequent roots to 0.3m

- below 0.5m, brown, moist

- below 1.8m, trace clay

CLAY - and silt, medium plastic, moist, soft to firm, brown

CLAY (SHALE) - silty, medium plastic, moist, very stiff, grey

TEST HOLE TERMINATED AT 4.6m BELOW EXISTING GRADE
Caving was observed from surface to 2.4m during drilling.
No seepage or sloughing was observed during drilling.
Test hole remained open to 0.2m with no water accumulated above the caved material prior to backfilling.
Test hole backfilled with auger cuttings and bentonite.

WSP Canada Inc.	LOGGED BY: JC	COMPLETION DEPTH: 4.6 m
	REVIEWED BY: NB	COMPLETION DATE: August 2, 2024
	Figure No. A13	Sheet 1 of 1

PROJECT: Boissevain Borrow Source	DRILLER: Paddock Drilling Ltd.	TEST HOLE ID: **TH24-24**
CLIENT: Manitoba Water Services Board	DRILL RIG: Mobile B48	PROJECT No: CA0017817.1877
LOCATION: NW13-3-20W	DRILL METHOD: 125mm SSA	ELEVATION: Not Surveyed

SAMPLE TYPE: ▨ Shelby Tube ▧ No Recovery ☒ SPT (N) ☰ Grab Sample ▥ Split-Pen ◧ Core

BACKFILL TYPE: ■ Bentonite ⁘ Pea Gravel ▧ Drill Cuttings ⁘ Grout ▥ Slough ⁘ Sand

DEPTH (m)	▲ UNCONFINED COMPRESSION (kPa) ▲ 100 200 300 400 ☒ POCKET PENETROMETER (kPa) ☒ 100 200 300 400 ★ TORVANE (kPa) ★ 50 100 150 200 PLASTIC M.C. LIQUID 20 40 60 80	SOIL SYMBOL	MUSCS	SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NO	SPT (N)	COMMENTS	DEPTH (m)
0		⁘	SP	SAND - trace silt, poorly graded, fine grained, damp to moist, brown, occasional grass roots to 0.2m	☰	1			
		⁘		- below 0.5m, brown	☰	2			
1		⁘			☰	3			1
2		⁘			☰	4			2
3		⁘		- below 3.0m, compact (inferred)	☰	5			3
4		⁘			☰	6			4
				TEST HOLE TERMINATED AT 4.6m BELOW EXISTING GRADE Caving was observed below 4.4m during drilling. No sloughing or seepage was observed during drilling. Test hole remained open to 4.4m with no water accumulated above the caved material prior to backfilling. Test hole backfilled with auger cuttings and bentonite.					
5									5
6									6
7									

WSP Canada Inc.

LOGGED BY: JC	COMPLETION DEPTH: 4.6 m
REVIEWED BY: NB	COMPLETION DATE: August 2, 2024
Figure No. A14	Sheet 1 of 1

APPENDIX

C

LABORATORY TEST REPORTS

MOISTURE CONTENT OF SOIL AND ROCK
(ASTM D2216)

Client: WSP Canada Inc.
Project: Boissevain Lagoon
Site Location: Onsite
Date Sampled: May 08, 2024
Sampled By: John Catigay

Lab No.: CA0017817.1877-MC
Project No.: CA0017817.1877
Report Date: Aug 02, 2023
Date Tested: Jun 05, 2024
Tested By: Aminat Akande

Test Hole No.	Sample No.	Depth (ft)	Moisture Content (%)
TH24-01	S1	1.0	12.5
TH24-01	S2	2.5	15.1
TH24-01	S3	5.0	18.7
TH24-01	S4	7.0	18.1
TH24-01	S5	10.0	17.9
TH24-01	S6	10-10.5	19.3
TH24-01	S8	20-20.75	11.3
TH24-01	S9	25-26.5	11.8
TH24-01	S10	30-31.5	24.6
TH24-01	S11	30-36.5	21.8
TH24-01	S12	40-41.5	22.1
TH24-02	S1	1.0	18.4
TH24-02	S2	2.5	15.9
TH24-02	S3	4.0	15.4
TH24-02	S4	7.5	18.6
TH24-02	S5	10.5	16.1
TH24-02	S6	10-11.5	18.6
TH24-02	S8	20-21.5	10.4
TH24-02	S9	25-26.5	23.2
TH24-02	S10	30-31.5	22.7
TH24-02	S11	35-36.5	22.6
TH24-02	S12	40-41.5	21.1
TH24-03	S1	1.0	28.6
TH24-03	S2	2.5	30.7
TH24-03	S3	4.5	20.5
TH24-03	S4	5-6.5	26.6
TH24-03	S5	7.5	25.6
TH24-03	S6	8.0	15.4
TH24-03	S7	10-11.5	15.7
TH24-03	S8	12.5	17.3
TH24-03	S9	15.0	19.0
TH24-03	S10	15-16.5	25.5
TH24-03	S11	15.5	19.2
TH24-03	S12	18.0	18.3
TH24-03	S13	20.0	14.2
TH24-03	S14	20-21.5	9.7
TH24-04	S1	1.0	30.8
TH24-04	S2	2.5	24.8
TH24-04	S3	5.0	25.0
TH24-04	S4	6.0	14.8
TH24-04	S5	7.5	15.1
TH24-04	S6	10.0	20.1
TH24-04	S7	10-11.5	21.4
TH24-04	S8	15.0	18.1
TH24-04	S10	16.0	16.9
TH24-04	S11	20.0	16.7

Test Hole No.	Sample No.	Depth (ft)	Moisture Content (%)
TH24-05	S1	0.5	38.7
TH24-05	S2	1.0	31.3
TH24-05	S3	2.5	31.1
TH24-05	S4	5.0	30.5
TH24-05	S6	7.5	25.2
TH24-05	S7	7.5	27.1
TH24-05	S9	13.5	10.7
TH24-05	S10	15.0	13.9
TH24-05	S11	16-16.5	14.5
TH24-05	S12	20.0	15.2
TH24-05	S13	20-21.5	17.9
TH24-06	S1	1.0	49.9
TH24-06	S2	2.5	25.1
TH24-06	S3	4.5	13.9
TH24-06	S4	5-6.5	14.1
TH24-06	S5	7.5	15.2
TH24-06	S6	10.0	16.5
TH24-06	S7	10-11.5	15.2
TH24-06	S8	12.5	16.4
TH24-06	S9	15.0	17.8
TH24-06	S10	15-16.5	15.1
TH24-06	S11	20.0	25.0
TH24-06	S12	20-21.5	12.6
TH24-07	S1	1.0	39.8
TH24-07	S2	2.5	28.9
TH24-07	S3	5.0	25.0
TH24-07	S4	5-6.5	22.1
TH24-07	S5	7.0	14.1
TH24-07	S6	9.5	21.3
TH24-07	S7	10-11.5	20.9
TH24-07	S8	15.0	16.9
TH24-07	S9	15-16.5	8.2
TH24-07	S10	15.5	15.3
TH24-07	S11	20.0	18.6
TH24-08	S1	1.0	34.4
TH24-08	S2	2.5	25.7
TH24-08	S3	5.0	15.3
TH24-08	S4	5-6.5	17.5
TH24-08	S5	7.5	16.9
TH24-08	S6	10.0	20.2
TH24-08	S7	10-11.5	27.0
TH24-08	S8	15.0	22.9
TH24-08	S9	15.5	17.8
TH24-08	S10	20.0	16.9

Reviewed by: [redacted]

Md Nazri Mohidin, E.I.T
Experienced Laboratory Technician

ATTERBERG LIMITS
(ASTM D4318)

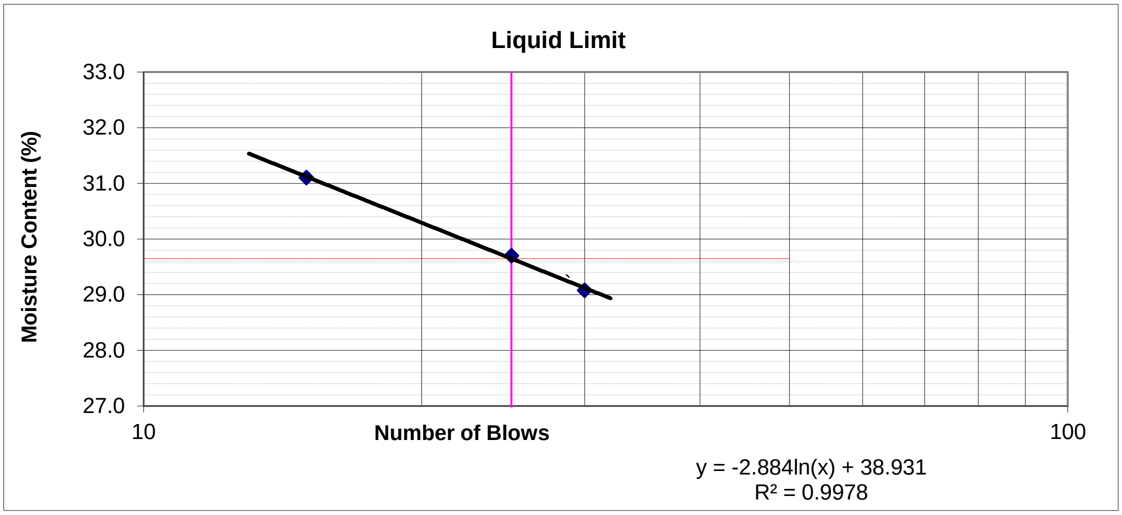
Client:	WSP Canada Inc.	**Lab No.:**	-
Project:	Boissevain Lagoon	**Project No.:**	CA0017817.1877
Site Location:	Boissevain	**Report Date:**	June 5, 2024
Date Sampled:	May 8, 2024	**Date Tested:**	June 3, 2024
Sampled By:	John Catigay	**Date Received:**	May 8, 2024

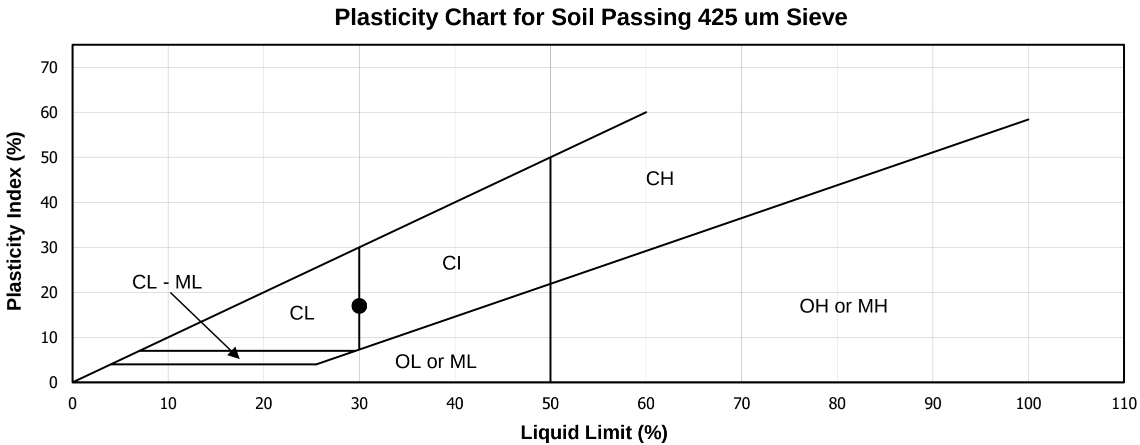
Testhole No.: TH24-01 **Sample No.:** S07 **Depth (ft):** 15

Drying Method: Oven Dried **Method:** Multi-Point **Tested By:** Tuong Lam

USCS Symbol: CI
Plastic Limit (%): 13
Liquid Limit (%): 30
Plasticity Index: **17**

	Liquid Limit			Plastic Limit	
	1	2	3	1	2
Wt. Of TARE	15.47	15.48	10.94	15.20	14.88
Wt. Of WET Sample and Tare	28.12	29.41	23.50	22.84	21.99
Wt. Of DRY Sample and Tare	25.27	26.22	20.52	21.94	21.16
Wt. Of WATER	2.85	3.19	2.98	0.90	0.83
Wt. Of DRY SAMPLE	9.80	10.74	9.58	6.74	6.28
PERCENT MOISTURE	**29.08**	**29.70**	**31.11**	**13.35**	**13.22**
Number of Blows	30	25	15		





Comments:

Reviewed by: [redacted]

Md Nazri Mohidin, E.I.T
Experienced Laboratory Technician

The test data given herein pertain to the sample provided. Reporting of these data constitutes a testing service. Engineering review and interpretation may be provided upon written request.

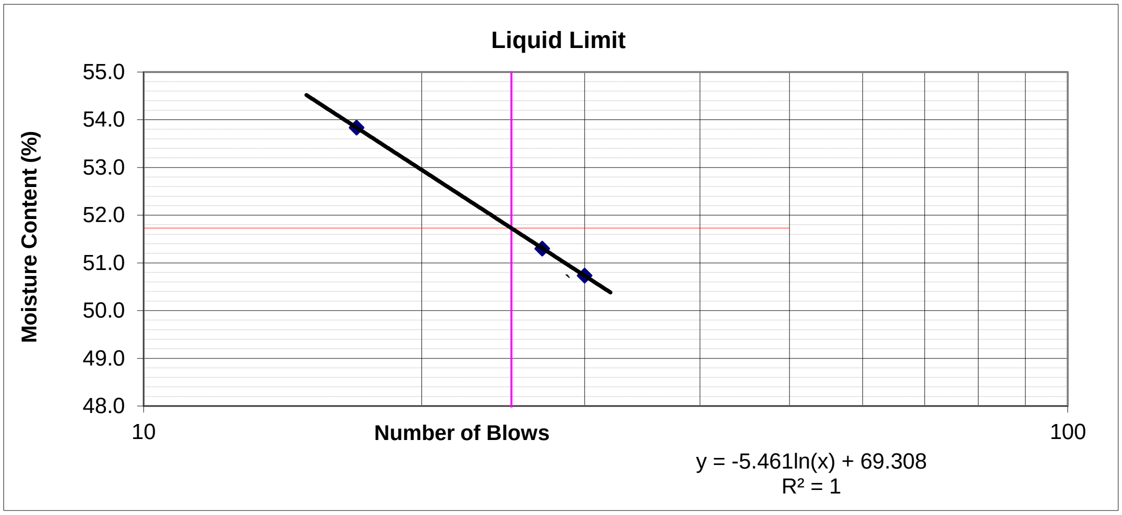
ATTERBERG LIMITS
(ASTM D4318)

Client:	WSP Canada Inc.	**Lab No.:**	-
Project:	Boissevain Lagoon	**Project No.:**	CA0017817.1877
Site Location:	Boissevain	**Report Date:**	June 5, 2024
Date Sampled:	May 8, 2024	**Date Tested:**	June 3, 2024
Sampled By:	John Catigay	**Date Received:**	May 8, 2024

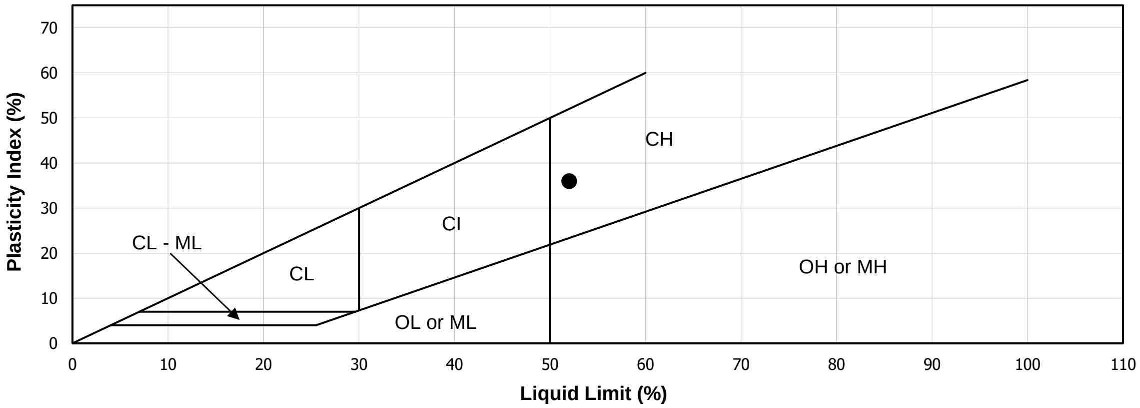
Testhole No.:	TH24-02	**Sample No.:**	S09	**Depth (ft):**	25-26.5
Drying Method:	Oven Dried	**Method:**	Multi-Point	**Tested By:**	Tuong Lam

USCS Symbol:	CH
Plastic Limit (%):	16
Liquid Limit (%):	52
Plasticity Index:	**36**

	Liquid Limit			**Plastic Limit**	
	1	**2**	**3**	**1**	**2**
Wt. Of TARE	18.45	18.11	13.39	13.58	11.58
Wt. Of WET Sample and Tare	30.72	31.47	24.62	20.90	18.71
Wt. Of DRY Sample and Tare	26.59	26.94	20.69	19.87	17.71
Wt. Of WATER	4.13	4.53	3.93	1.03	1.00
Wt. Of DRY SAMPLE	8.14	8.83	7.30	6.29	6.13
PERCENT MOISTURE	**50.74**	**51.30**	**53.84**	**16.38**	**16.31**
Number of Blows	30	27	17		



Plasticity Chart for Soil Passing 425 um Sieve



Comments: ______________________

Reviewed by: [redacted]

Md Nazri Mohidin, E.I.T
Experienced Laboratory Technician

ATTERBERG LIMITS
(ASTM D4318)

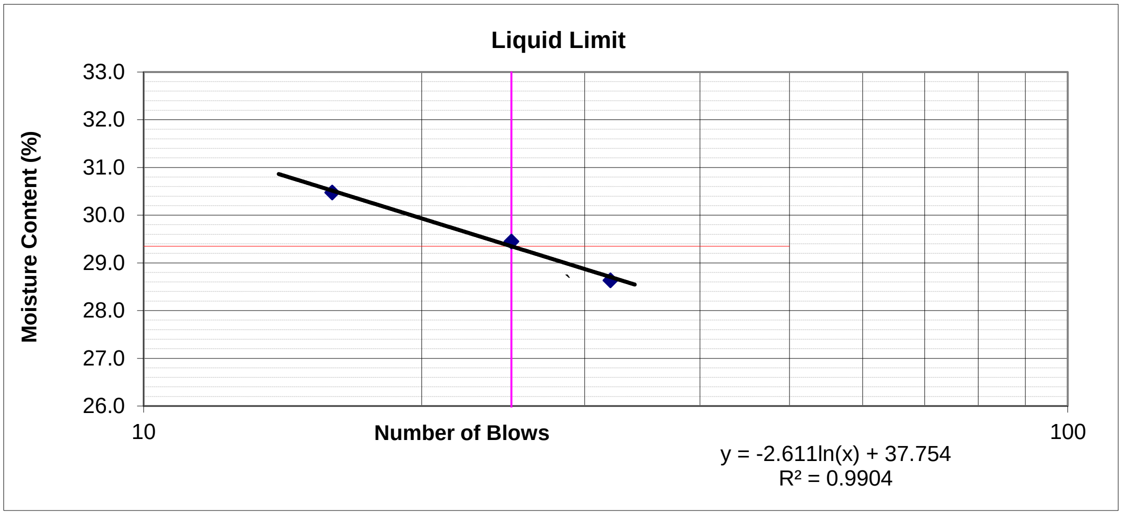
Client:	WSP Canada Inc.	**Lab No.:**	-
Project:	Boissevain Lagoon	**Project No.:**	CA0017817.1877
Site Location:	Boissevain	**Report Date:**	June 5, 2024
Date Sampled:	May 8, 2024	**Date Tested:**	June 3, 2024
Sampled By:	John Catigay	**Date Received:**	May 8, 2024

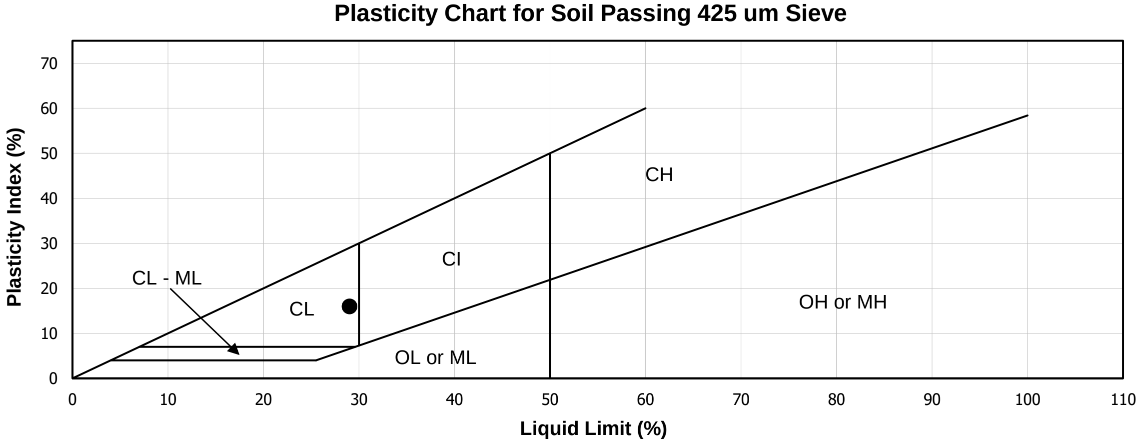
Testhole No.: TH24-03 **Sample No.:** S13 **Depth (ft):** 19-20

Drying Method: Oven Dried **Method:** Multi-Point **Tested By:** Tuong Lam

USCS Symbol: CL
Plastic Limit (%): 13
Liquid Limit (%): 29
Plasticity Index: **16**

	Liquid Limit			Plastic Limit	
	1	2	3	1	2
Wt. Of TARE	14.26	14.89	15.41	15.37	20.20
Wt. Of WET Sample and Tare	28.14	30.23	27.44	22.63	27.87
Wt. Of DRY Sample and Tare	25.05	26.74	24.63	21.82	27.01
Wt. Of WATER	3.09	3.49	2.81	0.81	0.86
Wt. Of DRY SAMPLE	10.79	11.85	9.22	6.45	6.81
PERCENT MOISTURE	**28.64**	**29.45**	**30.48**	**12.56**	**12.63**
Number of Blows	32	25	16		





Comments: ____________________

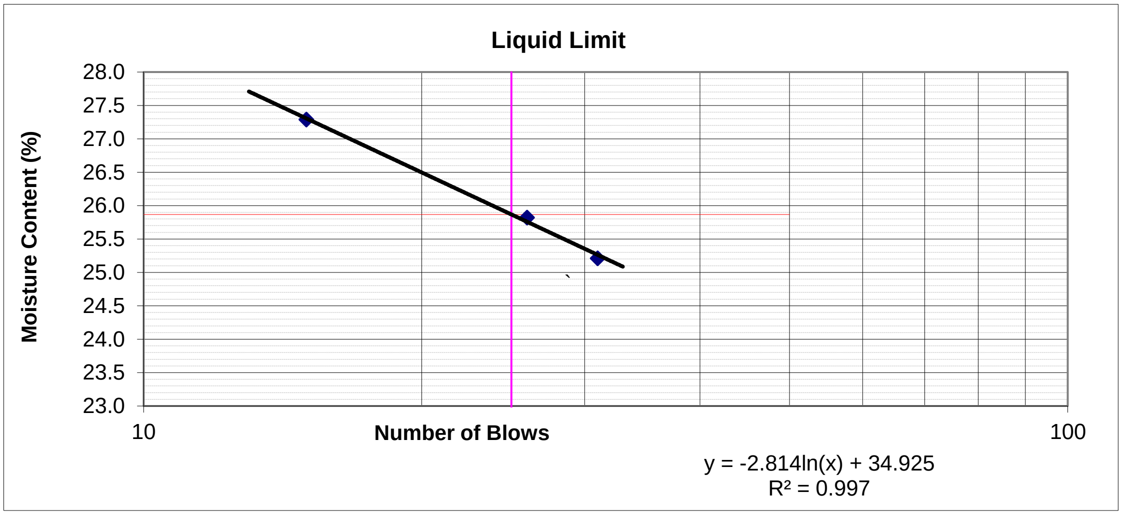
Reviewed by: [redacted]
Md Nazri Mohidin, E.I.T
Experienced Laboratory Technician

ATTERBERG LIMITS
(ASTM D4318)

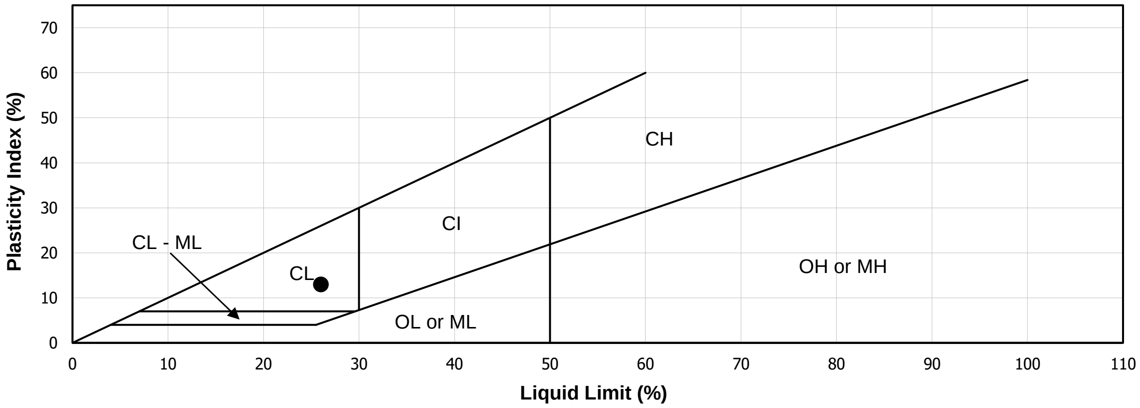
Client: WSP Canada Inc. **Lab No.:** -
Project: Boissevain Lagoon **Project No.:** CA0017817.1877
Site Location: Boissevain **Report Date:** June 5, 2024
Date Sampled: May 8, 2024 **Date Tested:** June 3, 2024
Sampled By: John Catigay **Date Received:** May 8, 2024
Testhole No.: TH24-04 **Sample No.:** S10 **Depth (ft):** 15.5-16
Drying Method: Oven Dried **Method:** Multi-Point **Tested By:** Tuong Lam

USCS Symbol: CL
Plastic Limit (%): 13
Liquid Limit (%): 26
Plasticity Index: **13**

	Liquid Limit			Plastic Limit	
	1	2	3	1	2
Wt. Of TARE	10.86	14.05	12.02	20.11	18.65
Wt. Of WET Sample and Tare	25.51	29.35	25.64	27.60	25.51
Wt. Of DRY Sample and Tare	22.56	26.21	22.72	26.73	24.72
Wt. Of WATER	2.95	3.14	2.92	0.87	0.79
Wt. Of DRY SAMPLE	11.70	12.16	10.70	6.62	6.07
PERCENT MOISTURE	**25.21**	**25.82**	**27.29**	**13.14**	**13.01**
Number of Blows	31	26	15		



Plasticity Chart for Soil Passing 425 um Sieve



Comments:

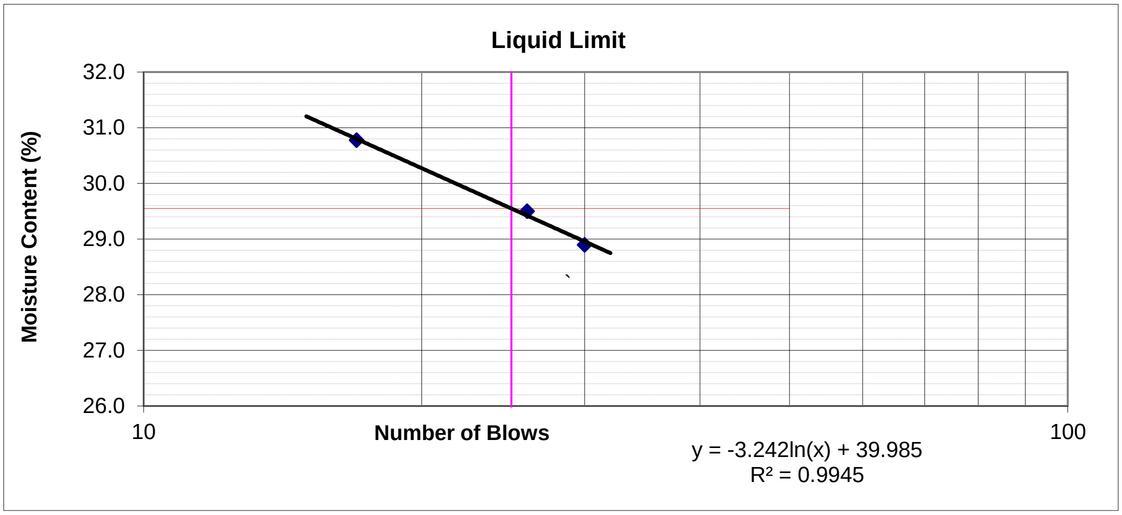
Reviewed by: [redacted]
Md Nazri Mohidin, E.I.T
Experienced Laboratory Technician

ATTERBERG LIMITS
(ASTM D4318)

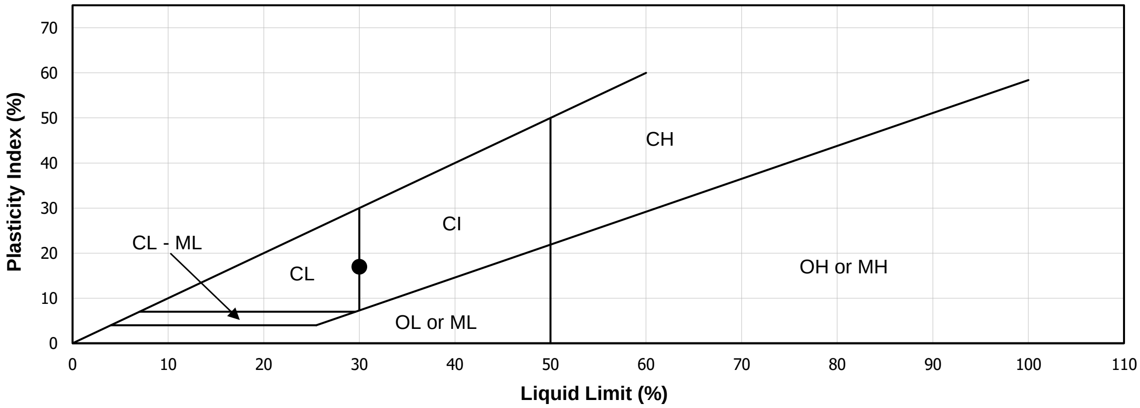
Client: WSP Canada Inc.
Lab No.: -
Project: Boissevain Lagoon
Project No.: CA0017817.1877
Site Location: Boissevain
Report Date: June 5, 2024
Date Sampled: May 8, 2024
Date Tested: June 3, 2024
Sampled By: John Catigay
Date Received: May 8, 2024
Testhole No.: TH24-05
Sample No.: S05
Depth (ft): 5-7
Drying Method: Oven Dried
Method: Multi-Point
Tested By: Tuong Lam

USCS Symbol:	CL
Plastic Limit (%):	13
Liquid Limit (%):	30
Plasticity Index:	**17**

	Liquid Limit			**Plastic Limit**	
	1	**2**	**3**	**1**	**2**
Wt. Of TARE	17.68	13.60	14.56	13.45	15.28
Wt. Of WET Sample and Tare	30.66	28.35	25.48	20.36	22.78
Wt. Of DRY Sample and Tare	27.75	24.99	22.91	19.55	21.90
Wt. Of WATER	2.91	3.36	2.57	0.81	0.88
Wt. Of DRY SAMPLE	10.07	11.39	8.35	6.10	6.62
PERCENT MOISTURE	**28.90**	**29.50**	**30.78**	**13.28**	**13.29**
Number of Blows	30	26	17		



Plasticity Chart for Soil Passing 425 um Sieve



Comments: ____________________

Reviewed by: [redacted]

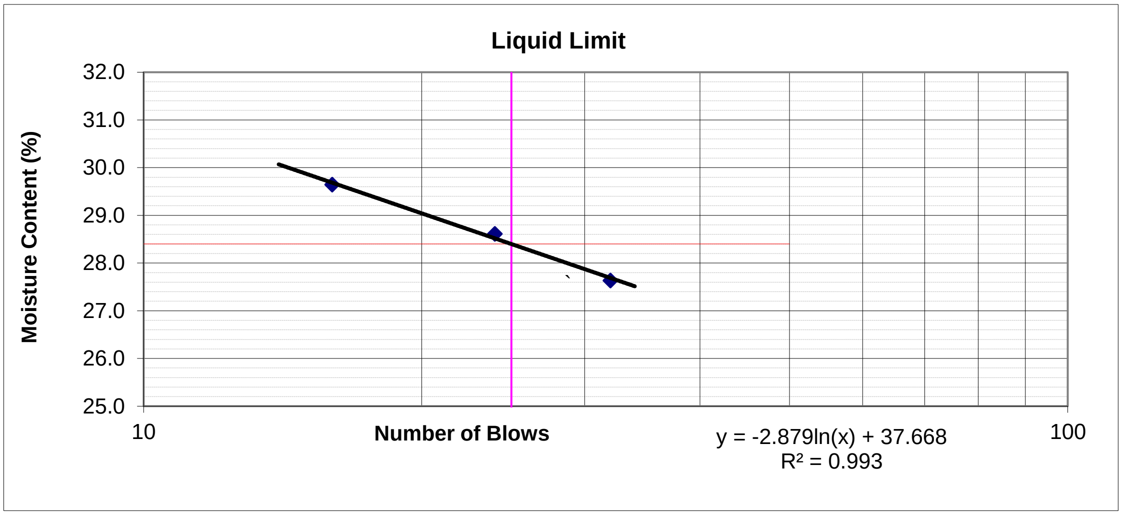
Md Nazri Mohidin, E.I.T
Experienced Laboratory Technician

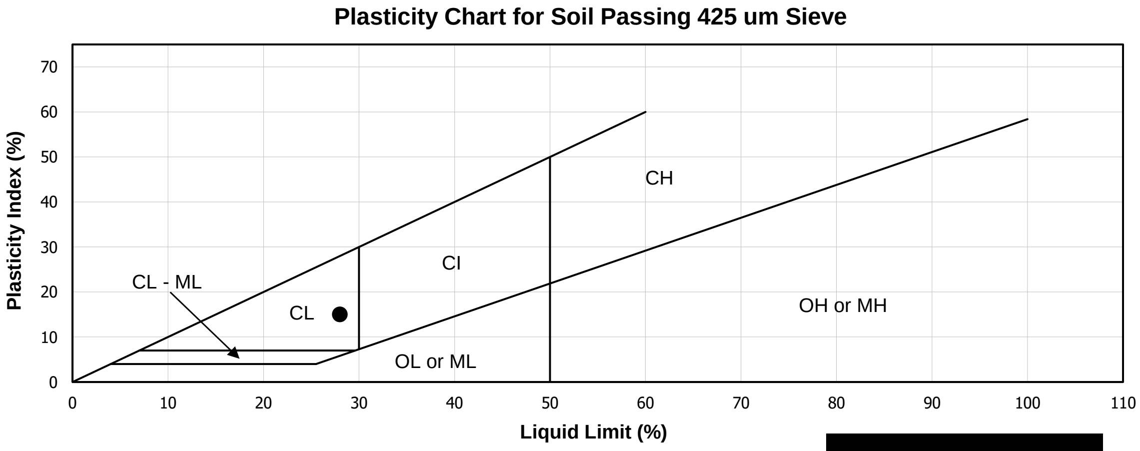
ATTERBERG LIMITS
(ASTM D4318)

Client: WSP Canada Inc.
Lab No.: -
Project: Boissevain Lagoon
Project No.: CA0017817.1877
Site Location: Boissevain
Report Date: June 5, 2024
Date Sampled: May 8, 2024
Date Tested: June 3, 2024
Sampled By: John Catigay
Date Received: May 8, 2024
Testhole No.: TH24-06
Sample No.: S09
Depth (ft): 14.5-15
Drying Method: Oven Dried
Method: Multi-Point
Tested By: Tuong Lam

USCS Symbol: CL
Plastic Limit (%): 13
Liquid Limit (%): 28
Plasticity Index: **15**

	Liquid Limit			Plastic Limit	
	1	2	3	1	2
Wt. Of TARE	15.33	15.48	12.30	15.39	15.51
Wt. Of WET Sample and Tare	28.91	28.74	25.77	22.42	23.53
Wt. Of DRY Sample and Tare	25.97	25.79	22.69	21.61	22.60
Wt. Of WATER	2.94	2.95	3.08	0.81	0.93
Wt. Of DRY SAMPLE	10.64	10.31	10.39	6.22	7.09
PERCENT MOISTURE	**27.63**	**28.61**	**29.64**	**13.02**	**13.12**
Number of Blows	32	24	16		





Comments:

Reviewed by: [redacted]

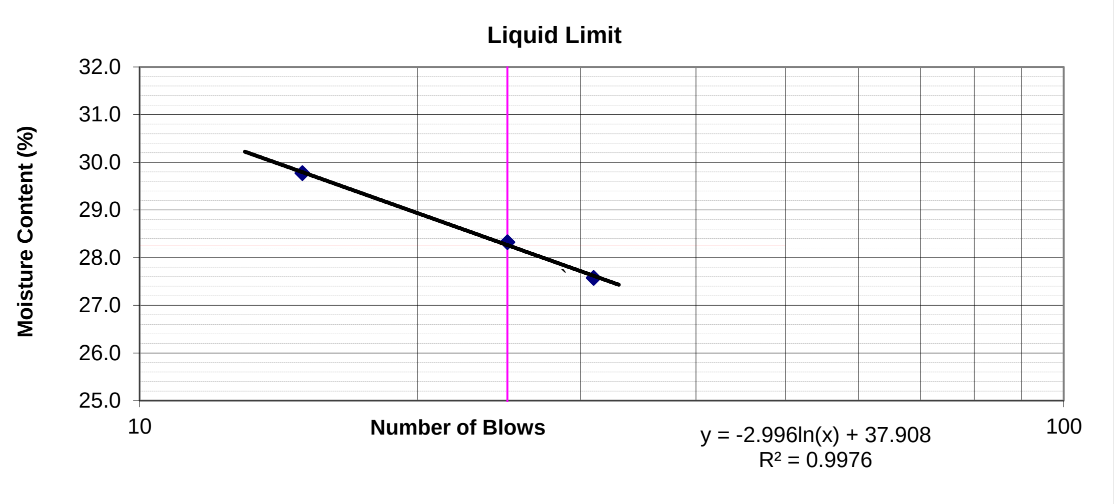
Md Nazri Mohidin, E.I.T
Experienced Laboratory Technician

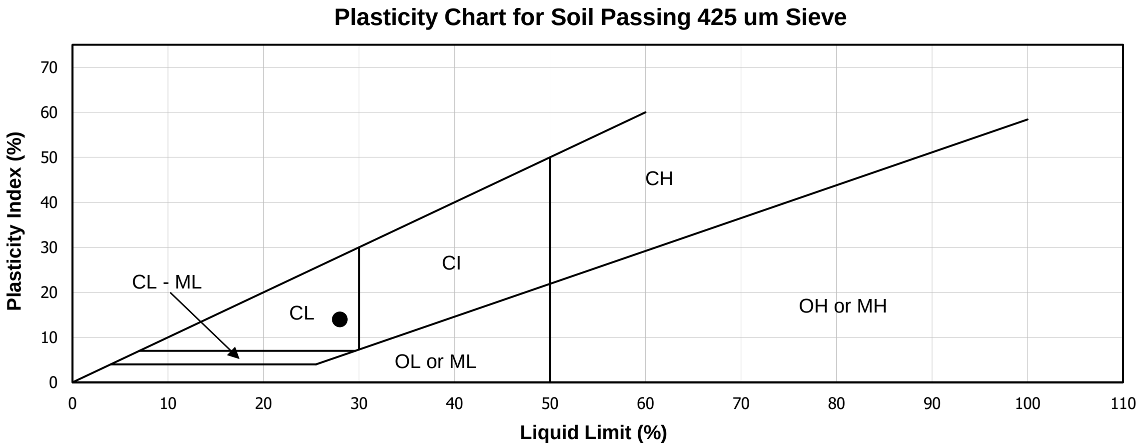
ATTERBERG LIMITS
(ASTM D4318)

Client: WSP Canada Inc.
Lab No.: -
Project: Boissevain Lagoon
Project No.: CA0017817.1877
Site Location: Boissevain
Report Date: June 5, 2024
Date Sampled: May 8, 2024
Date Tested: June 3, 2024
Sampled By: John Catigay
Date Received: May 8, 2024
Testhole No.: TH24-07
Sample No.: S11
Depth (ft): 19.5-20
Drying Method: Oven Dried
Method: Multi-Point
Tested By: Tuong Lam

USCS Symbol: CL
Plastic Limit (%): 14
Liquid Limit (%): 28
Plasticity Index: **14**

	Liquid Limit			**Plastic Limit**	
	1	**2**	**3**	**1**	**2**
Wt. Of TARE	14.68	10.64	15.34	15.47	10.16
Wt. Of WET Sample and Tare	30.04	26.27	29.20	23.25	17.34
Wt. Of DRY Sample and Tare	26.72	22.82	26.02	22.33	16.48
Wt. Of WATER	3.32	3.45	3.18	0.92	0.86
Wt. Of DRY SAMPLE	12.04	12.18	10.68	6.86	6.32
PERCENT MOISTURE	**27.57**	**28.33**	**29.78**	**13.41**	**13.61**
Number of Blows	31	25	15		





Comments:

Reviewed by: [redacted]

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Experienced Laboratory Technician

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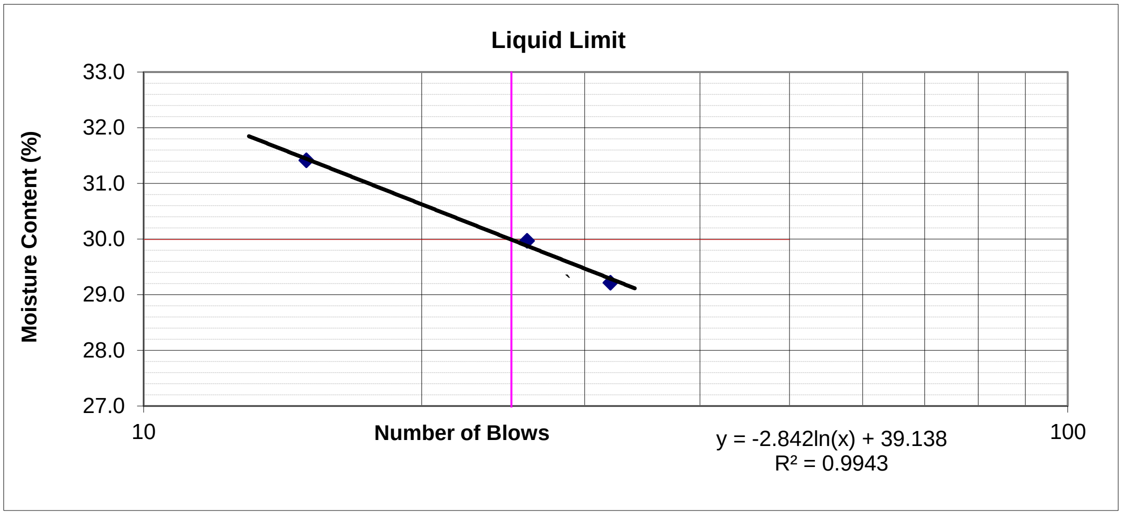
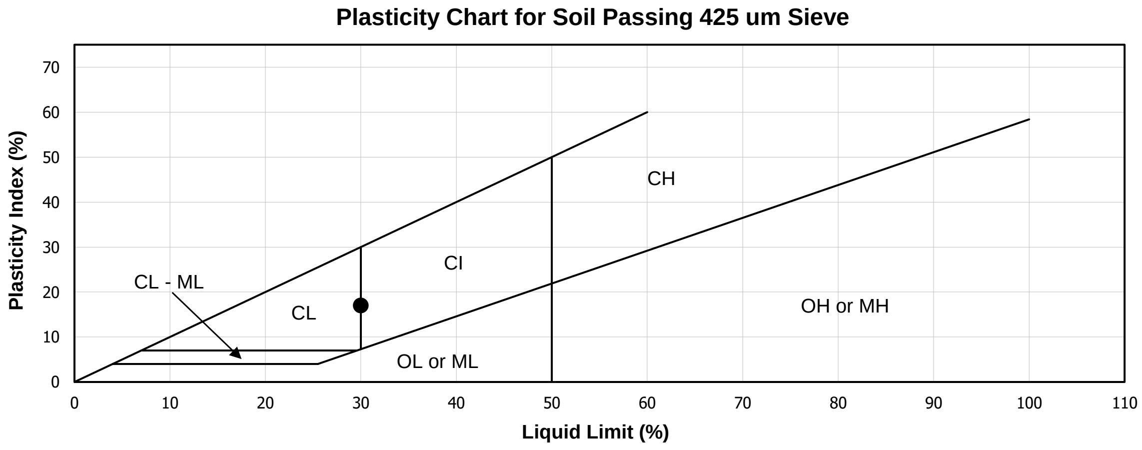
ATTERBERG LIMITS
(ASTM D4318)

Client: WSP Canada Inc.
Lab No.: -
Project: Boissevain Lagoon
Project No.: CA0017817.1877
Site Location: Boissevain
Report Date: June 5, 2024
Date Sampled: May 8, 2024
Date Tested: June 3, 2024
Sampled By: John Catigay
Date Received: May 8, 2024
Testhole No.: TH24-08
Sample No.: S10
Depth (ft): 19.5-20
Drying Method: Oven Dried
Method: Multi-Point
Tested By: Tuong Lam

USCS Symbol: CI
Plastic Limit (%): 13
Liquid Limit (%): 30
Plasticity Index: **17**

	Liquid Limit			**Plastic Limit**	
	1	**2**	**3**	**1**	**2**
Wt. Of TARE	15.51	10.17	15.30	15.44	11.57
Wt. Of WET Sample and Tare	30.06	23.70	28.77	22.99	19.11
Wt. Of DRY Sample and Tare	26.77	20.58	25.55	22.12	18.23
Wt. Of WATER	3.29	3.12	3.22	0.87	0.88
Wt. Of DRY SAMPLE	11.26	10.41	10.25	6.68	6.66
PERCENT MOISTURE	**29.22**	**29.97**	**31.41**	**13.02**	**13.21**
Number of Blows	32	26	15		





Comments:

Reviewed by: [redacted]
Md Nazri Mohidin, E.I.T
Experienced Laboratory Technician

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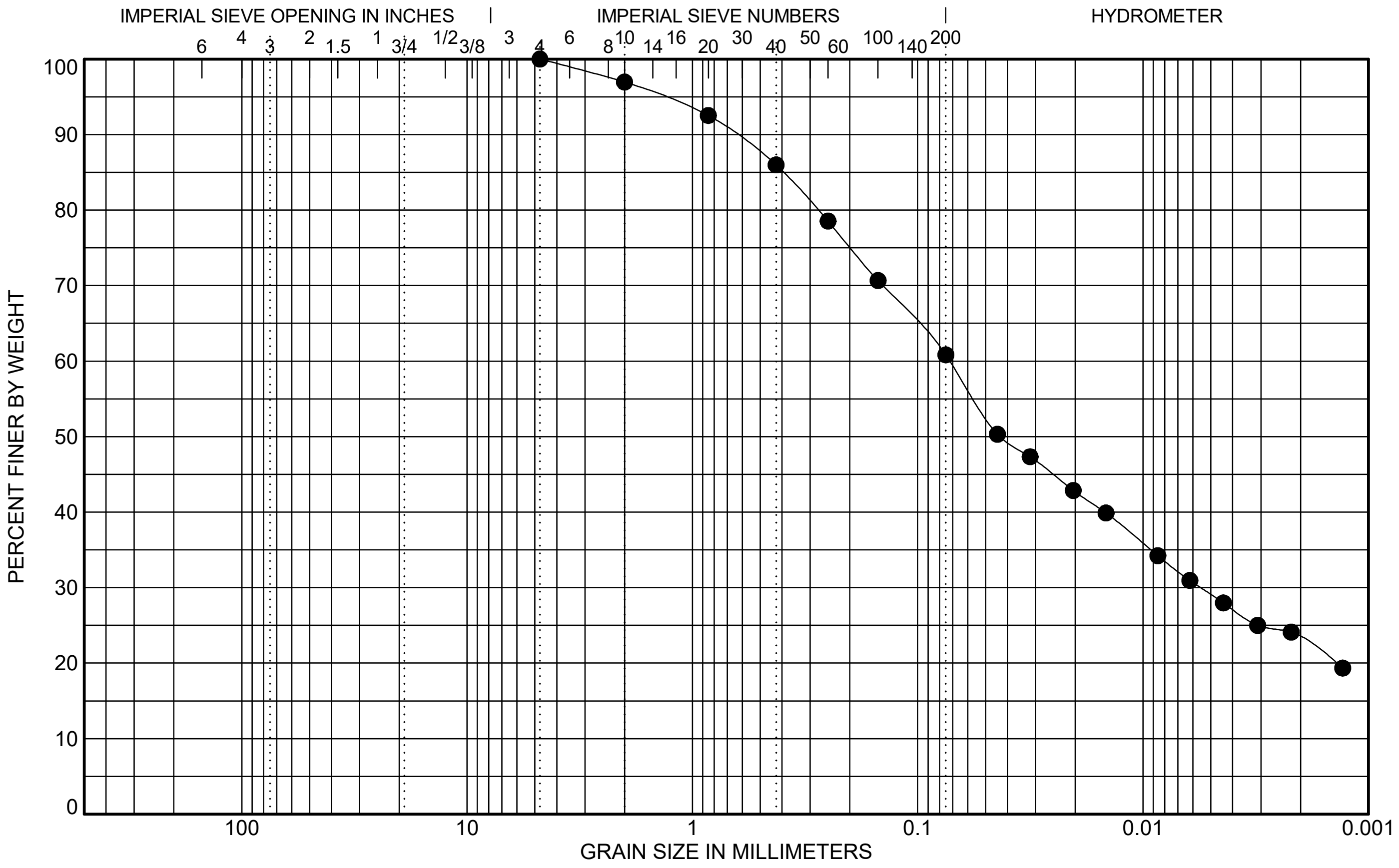
PARTICLE SIZE ANALYSIS

Report Date: 03 September 2024

Client		**Project**	
Name:	Manitoba Water Services Board	**Name:**	Boissevain Lagoon
Address:	Unit 1A – 2010 Currie Blvd	**Address:**	Boissevain
Attention:	Nathan Wittmeier	**Project No.:**	CA0017817.1877
PO Number:		**Manager:**	NB

Gradation Specification:



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	Sample ID	mUSCS	MC	D100	D60	D30	D10	Cc	Cu	% Gravel	% Sand	% Fines
●	TH24-01, 4.6 m	CI		4.8	0.1	0				0	39	38 (Silt) : 23 (Clay)

PARTICLE SIZE ANALYSIS

Report Date: 03 September 2024

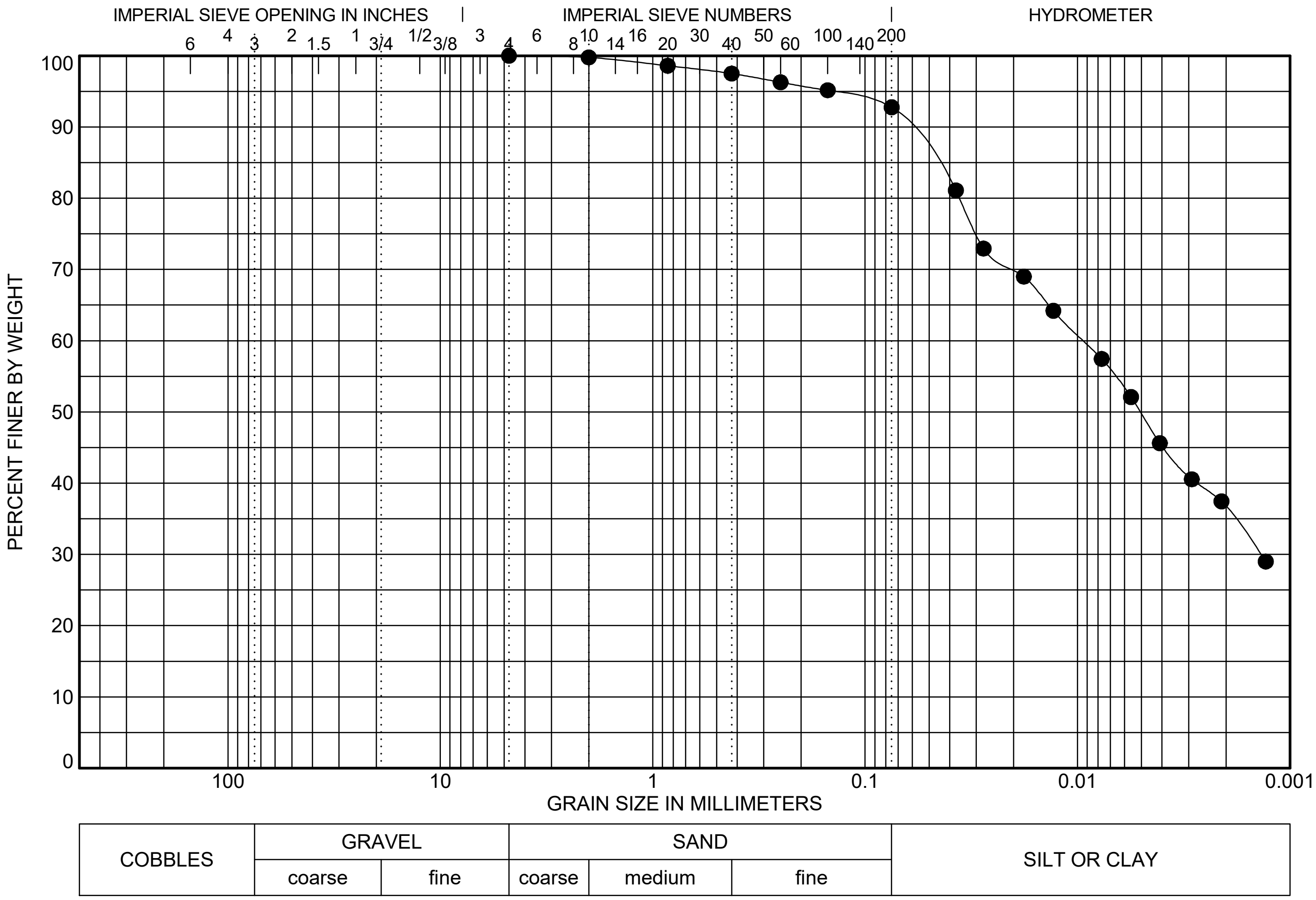
Client

Name: Manitoba Water Services Board
Address: Unit 1A – 2010 Currie Blvd
Attention: Nathan Wittmeier
PO Number:

Project

Name: Boissevain Lagoon
Address: Boissevain
Project No.: CA0017817.1877
Manager: NB

Gradation Specification:



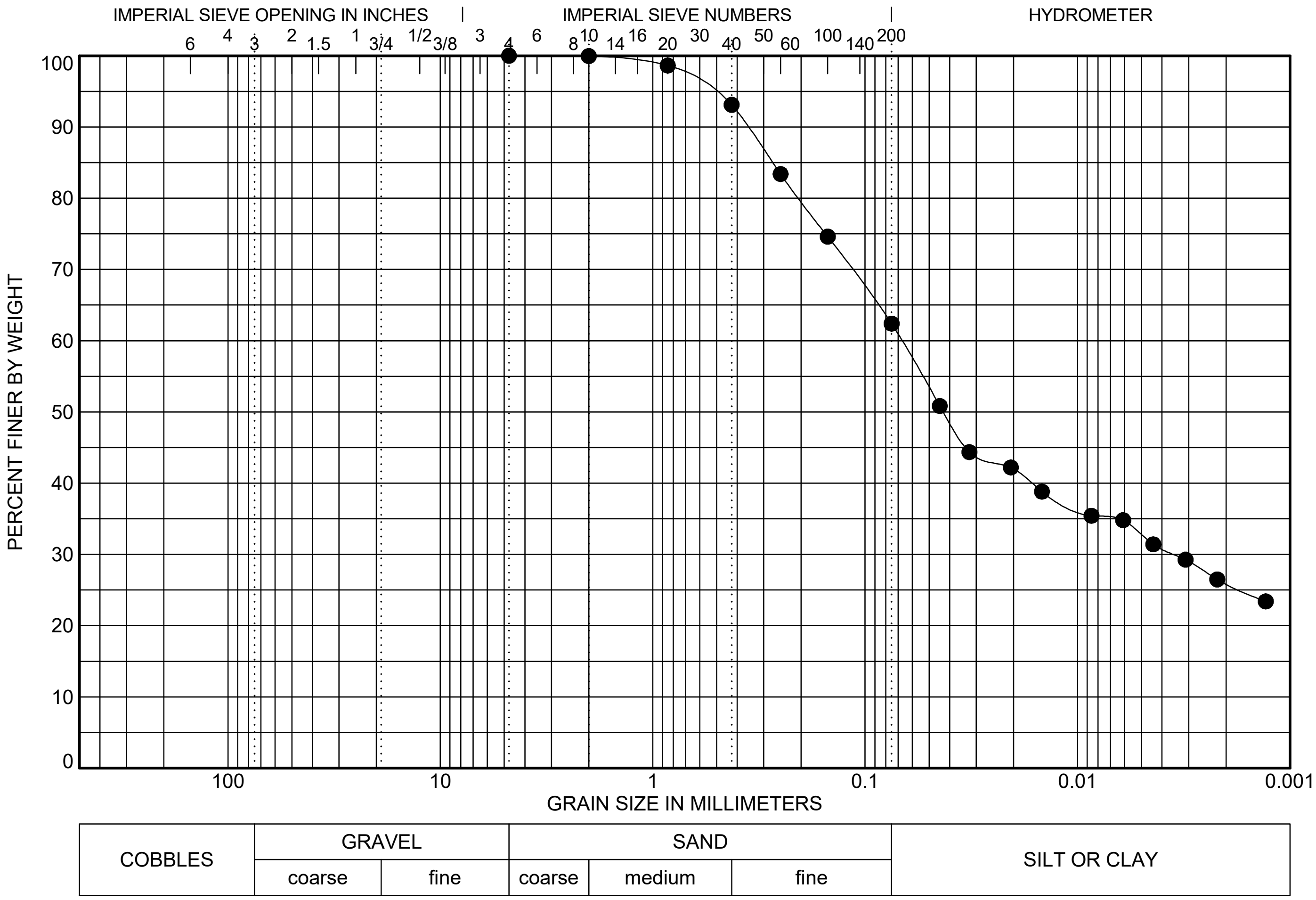
	Sample ID	mUSCS	MC	D100	D60	D30	D10	Cc	Cu	% Gravel	% Sand	% Fines
●	TH2402, 7.6 m	CH		4.8	0	0				0	7	56 (Silt) : 37 (Clay)

PARTICLE SIZE ANALYSIS

Report Date: 03 September 2024

Client		**Project**	
Name:	Manitoba Water Services Board	**Name:**	Boissevain Lagoon
Address:	Unit 1A – 2010 Currie Blvd	**Address:**	Boissevain
Attention:	Nathan Wittmeier	**Project No.:**	CA0017817.1877
PO Number:		**Manager:**	NB

Gradation Specification:



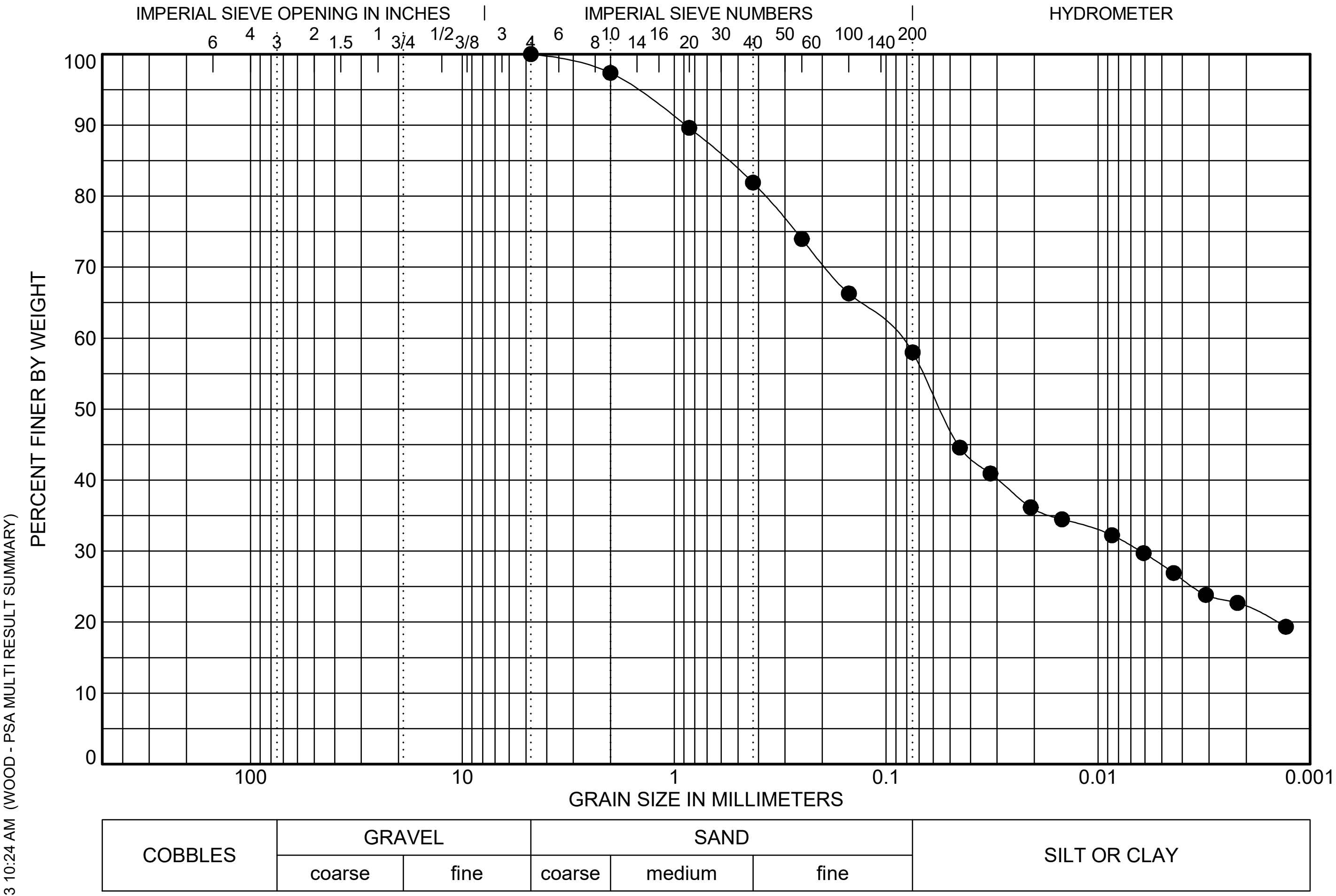
	Sample ID	mUSCS	MC	D100	D60	D30	D10	Cc	Cu	% Gravel	% Sand	% Fines
●	TH24-05, 1.5 m	CI		4.8	0.1	0				0	38	36 (Silt) : 26 (Clay)

PARTICLE SIZE ANALYSIS

Report Date: 03 September 2024

Client		**Project**	
Name:	Manitoba Water Services Board	**Name:**	Boissevain Lagoon
Address:	Unit 1A – 2010 Currie Blvd	**Address:**	Boissevain
Attention:	Nathan Wittmeier	**Project No.:**	CA0017817.1877
PO Number:		**Manager:**	NB

Gradation Specification:



	Sample ID	mUSCS	MC	D100	D60	D30	D10	Cc	Cu	% Gravel	% Sand	% Fines
●	TH24-07, 5.9 m	CL		4.8	0.1	0				0	42	36 (Silt) : 22 (Clay)

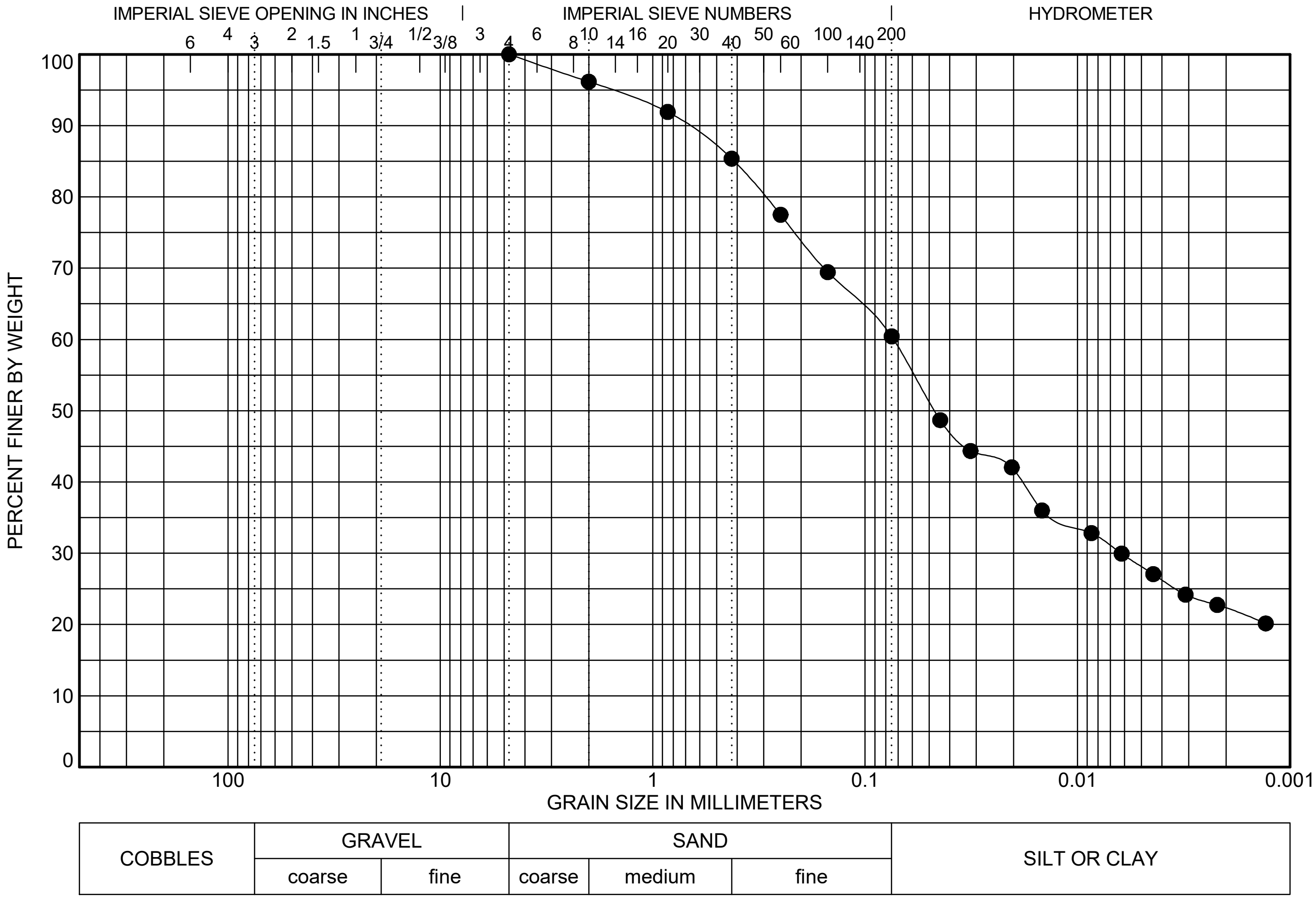
CA0017817 - BOISSEVAIN LAGOON HYDROMETER ONLY.GPJ 24/09/03 10:24 AM (WOOD - PSA MULTI RESULT SUMMARY)

PARTICLE SIZE ANALYSIS

Report Date: 03 September 2024

Client		**Project**	
Name:	Manitoba Water Services Board	**Name:**	Boissevain Lagoon
Address:	Unit 1A – 2010 Currie Blvd	**Address:**	Boissevain
Attention:	Nathan Wittmeier	**Project No.:**	CA0017817.1877
PO Number:		**Manager:**	NB

Gradation Specification:



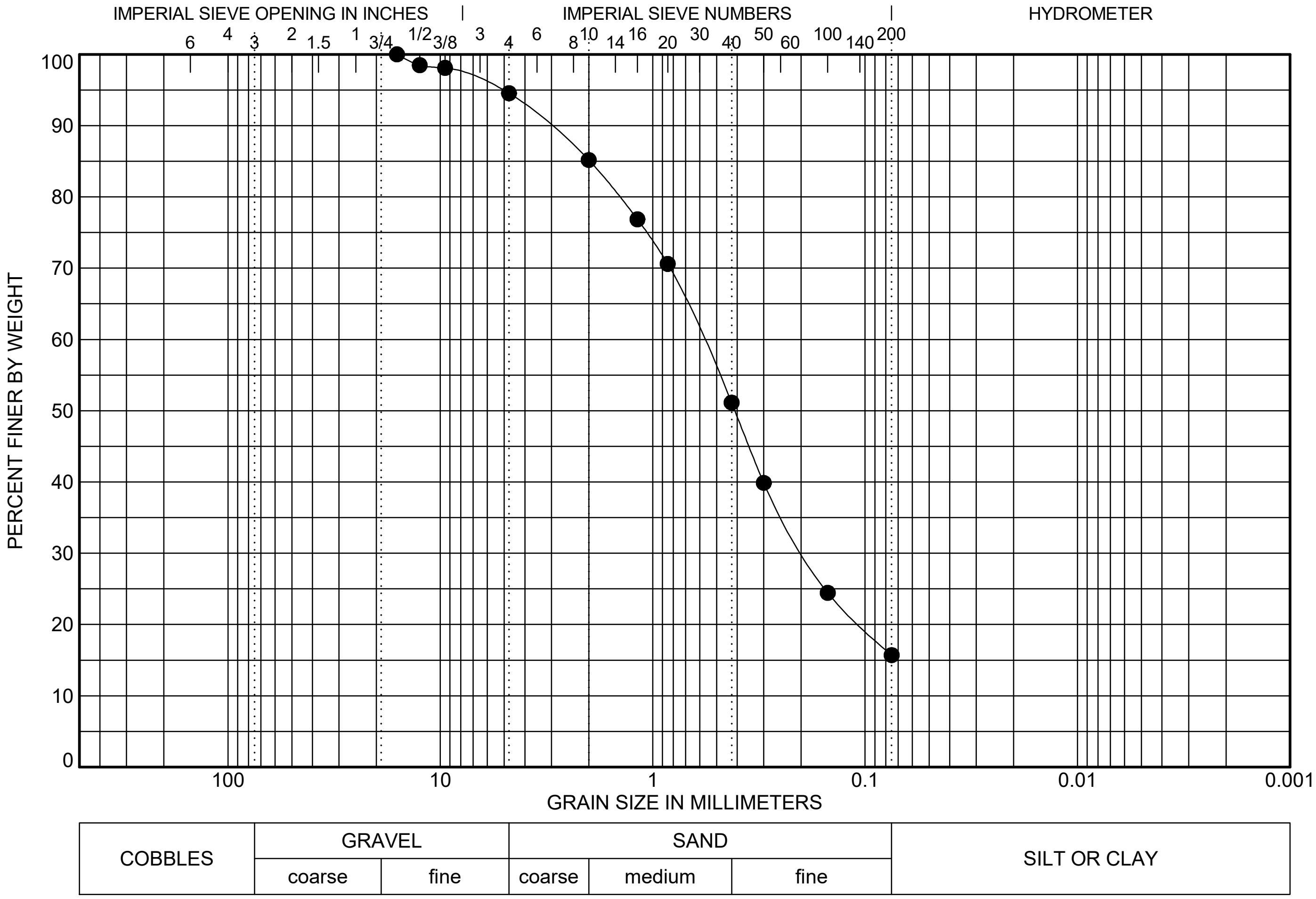
	Sample ID	mUSCS	MC	D100	D60	D30	D10	Cc	Cu	% Gravel	% Sand	% Fines
●	TH24-08, 5.9 m	CI		4.8	0.1	0				0	40	38 (Silt) : 22 (Clay)

PARTICLE SIZE ANALYSIS

Report Date: 03 September 2024

Client		**Project**	
Name:	Manitoba Water Services Board	**Name:**	Boissevain Lagoon
Address:	Unit 1A – 2010 Currie Blvd	**Address:**	Boissevain
Attention:	Nathan Wittmeier	**Project No.:**	CA0017817.1877
PO Number:		**Manager:**	NB

Gradation Specification:



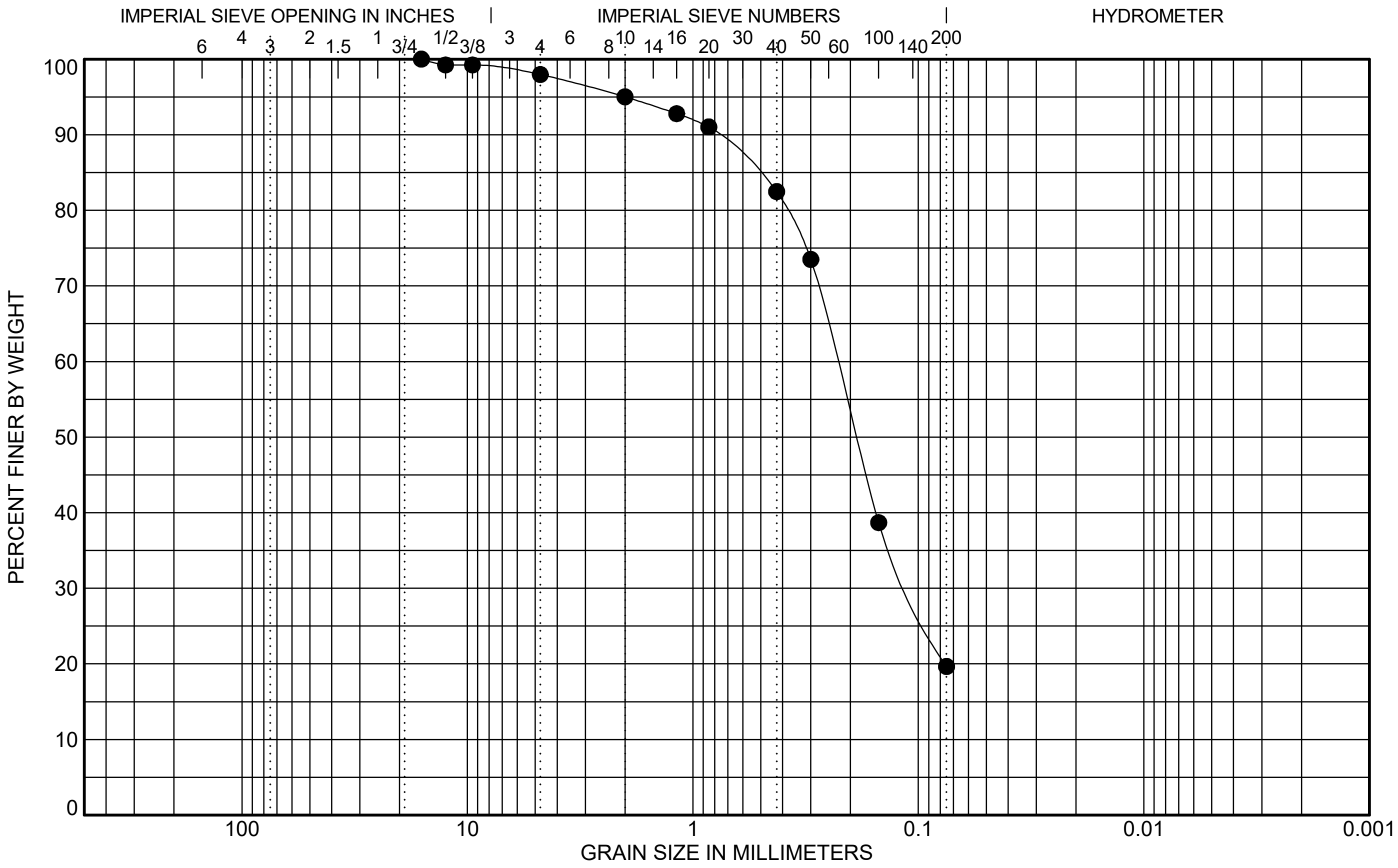
	Sample ID	mUSCS	MC	D100	D60	D30	D10	Cc	Cu	% Gravel	% Sand	% Fines
●	TH24-03, 4.6 m	SM		16	0.6	0.2				5	79	15.7

PARTICLE SIZE ANALYSIS

Report Date: 03 September 2024

Client		**Project**	
Name:	Manitoba Water Services Board	**Name:**	Boissevain Lagoon
Address:	Unit 1A – 2010 Currie Blvd	**Address:**	Boissevain
Attention:	Nathan Wittmeier	**Project No.:**	CA0017817.1877
PO Number:		**Manager:**	NB

Gradation Specification:



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

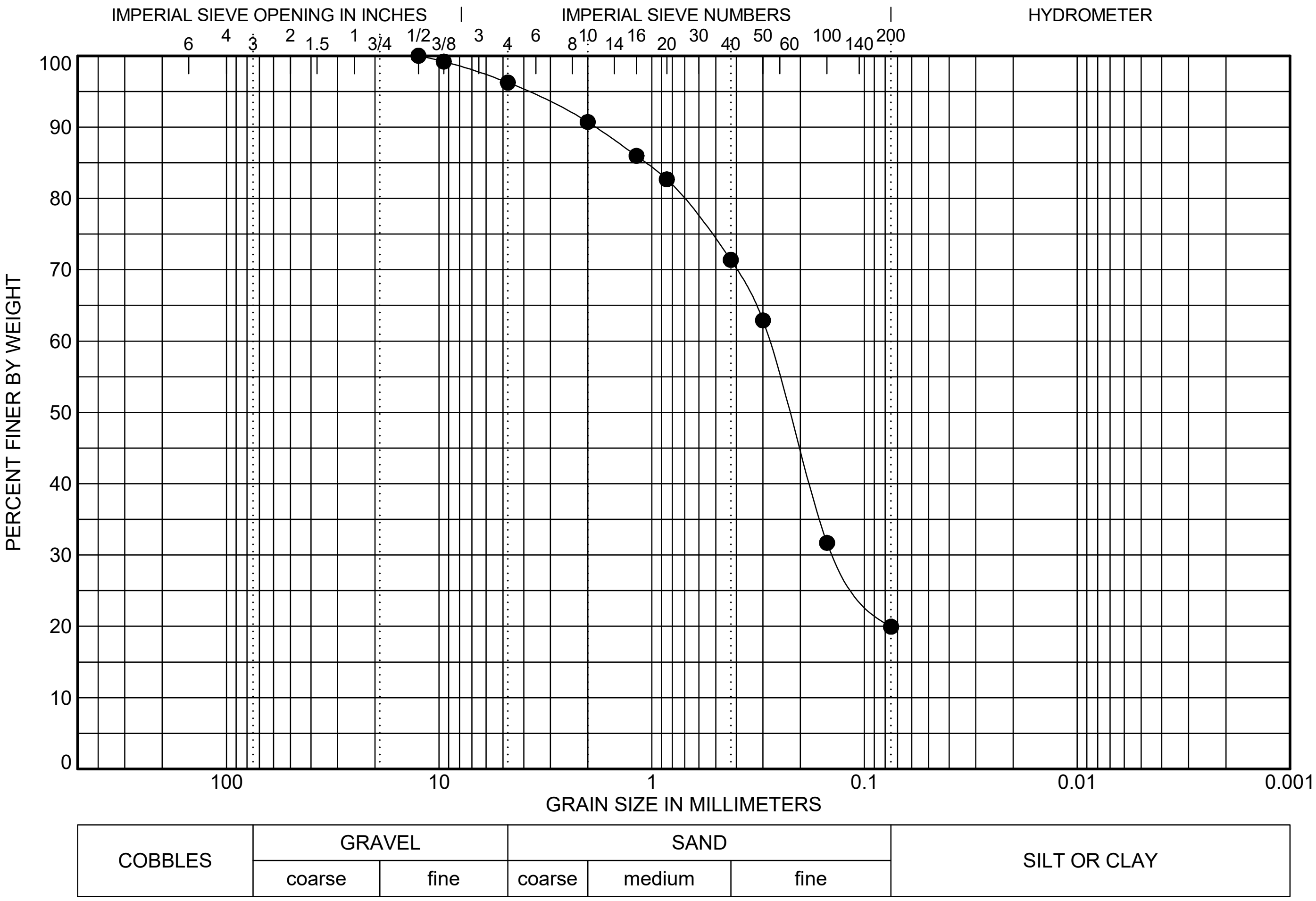
	Sample ID	mUSCS	MC	D100	D60	D30	D10	Cc	Cu	% Gravel	% Sand	% Fines
●	TH24-04, 2.9 m	SM		16	0.2	0.1				2	78	19.7

PARTICLE SIZE ANALYSIS

Report Date: 03 September 2024

Client		**Project**	
Name:	Manitoba Water Services Board	**Name:**	Boissevain Lagoon
Address:	Unit 1A – 2010 Currie Blvd	**Address:**	Boissevain
Attention:	Nathan Wittmeier	**Project No.:**	CA0017817.1877
PO Number:		**Manager:**	NB

Gradation Specification:



	Sample ID	mUSCS	MC	D100	D60	D30	D10	Cc	Cu	% Gravel	% Sand	% Fines
●	TH24-06, 2.9 m	SM		12.5	0.3	0.1				4	76	19.9

PARTICLE SIZE ANALYSIS

Report Date: 03 September 2024

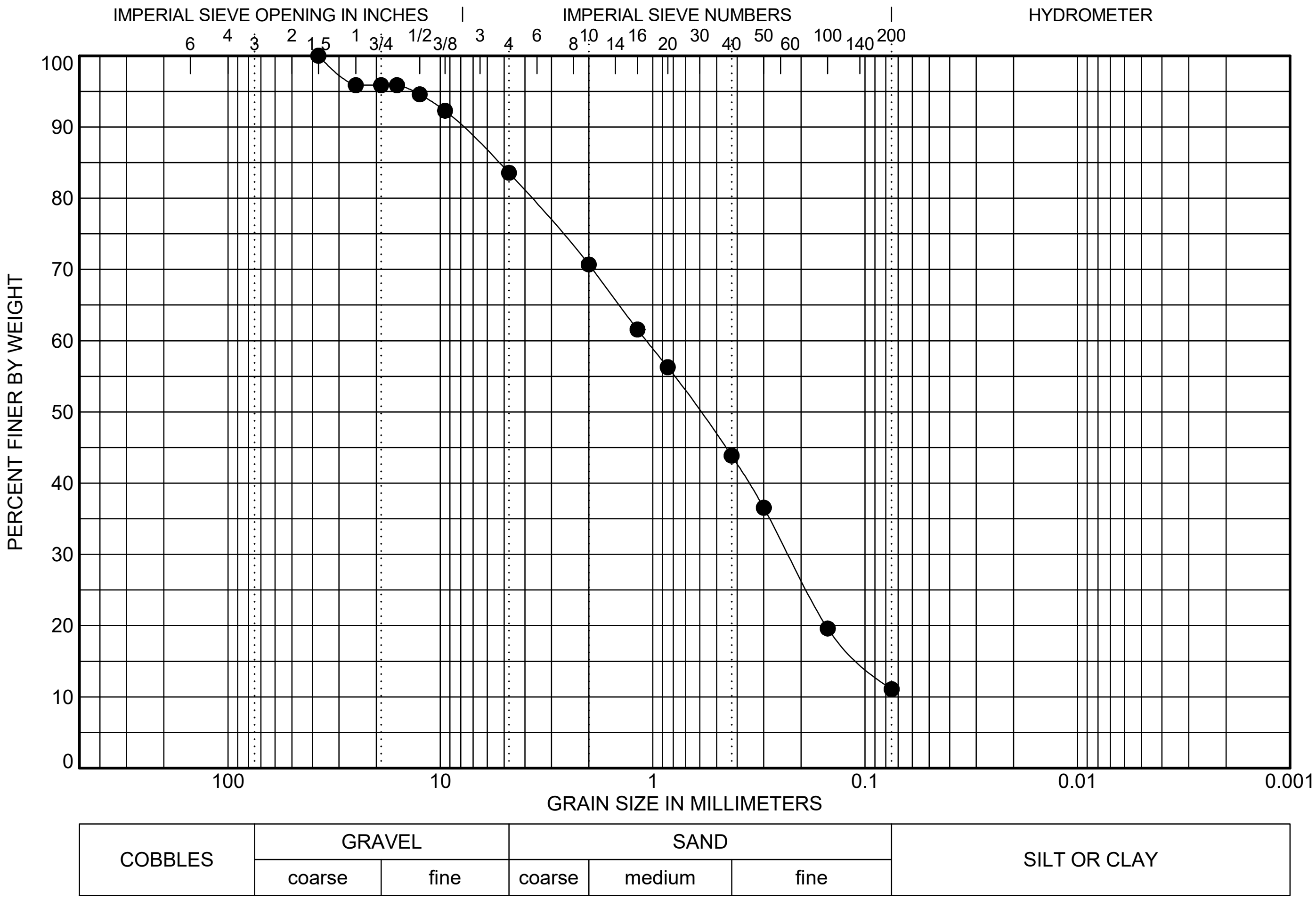
Client

Name: Manitoba Water Services Board
Address: Unit 1A – 2010 Currie Blvd
Attention: Nathan Wittmeier
PO Number:

Project

Name: Boissevain Lagoon
Address: Boissevain
Project No.: CA0017817.1877
Manager: NB

Gradation Specification:



	Sample ID	mUSCS	MC	D100	D60	D30	D10	Cc	Cu	% Gravel	% Sand	% Fines
●	TH24-08, 1.5 m	SP-SM		37.5	1.1	0.2		0.7	15.6	16	72	11.1

MOISTURE CONTENT OF SOIL AND ROCK

(ASTM D2216)

Client:	WSP Canada Inc.	**Lab No.:**	CA0017817.1877-MC03
Project:	Boissevain Lagoon	**Project No.:**	CA0017817.1877
Site Location:	Onsite	**Report Date:**	Aug 02, 2023
Date Sampled:	May 08, 2024	**Date Tested:**	Sep 17, 2024
Sampled By:	John Catigay	**Tested By:**	M'hand Ihya

Test Hole No.	Sample No.	Depth (ft)	Moisture Content (%)
TH24-11	S1	1.0	16.0
TH24-11	S2	2.5	23.4
TH24-11	S3	5.0	24.9
TH24-11	S4	7.0	28.0
TH24-11	S5	10.0	29.2

Reviewed by [redacted]

Md Nazri Mohidin, E.I.T
Experienced Laboratory Technician

Notice: The test data given herein pertain to the sample provided. Reporting of these data constitutes a testing service. Engineering review and interpretation may be provided upon written request.

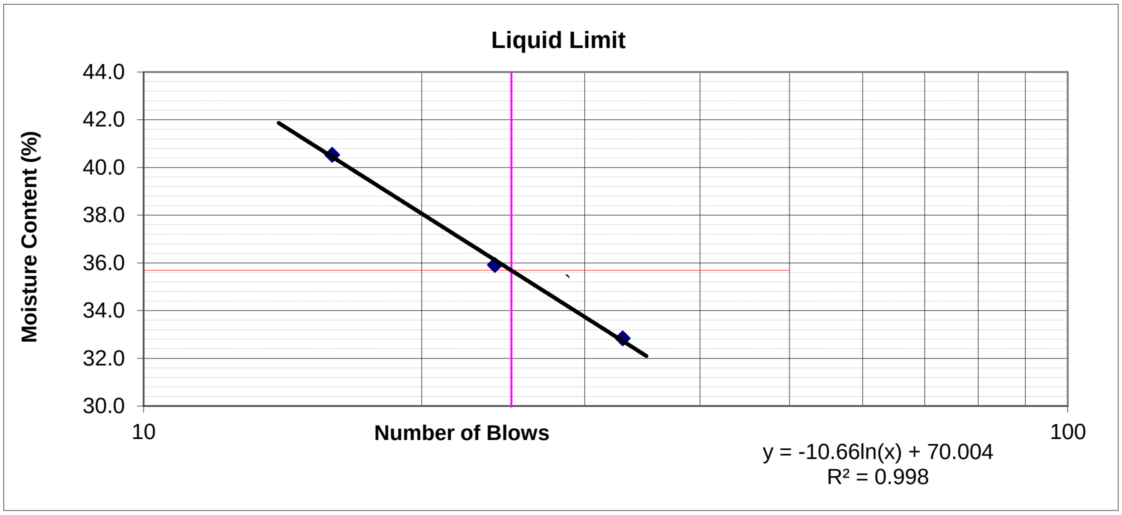
ATTERBERG LIMITS
(ASTM D4318)

Client: WSP Canada Inc.
Project: Boissevain Lagoon
Site Location: Boissevain
Date Sampled: August 9, 2024
Sampled By: Lith Trinh
Lab No.: -
Project No.: CA0017114.3097
Report Date: July 17, 2024
Date Tested: July 16, 2024
Date Received: July 5, 2024

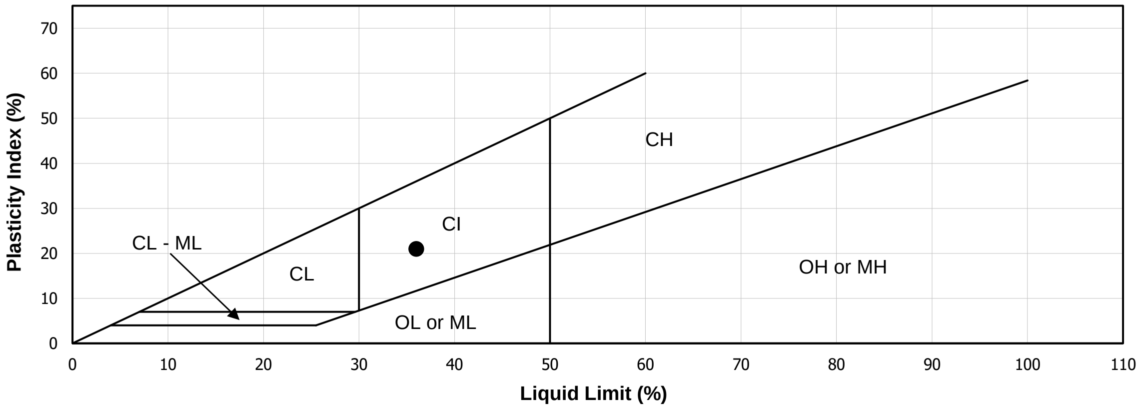
Testhole No.: TH24-09
Sample No.: Mixed
Depth (ft): -
Drying Method: Oven Dried
Method: Multi-Point
Tested By: Nazri Mohidin

USCS Symbol:	CI
Plastic Limit (%):	15
Liquid Limit (%):	36
Plasticity Index:	**21**

	Liquid Limit			Plastic Limit	
	1	2	3	1	2
Wt. Of TARE	6.92	8.94	7.16	13.34	13.59
Wt. Of WET Sample and Tare	10.48	12.27	10.87	21.57	22.14
Wt. Of DRY Sample and Tare	9.60	11.39	9.80	20.50	21.06
Wt. Of WATER	0.88	0.88	1.07	1.07	1.08
Wt. Of DRY SAMPLE	2.68	2.45	2.64	7.16	7.47
PERCENT MOISTURE	**32.84**	**35.92**	**40.53**	**14.94**	**14.46**
Number of Blows	33	24	16		



Plasticity Chart for Soil Passing 425 um Sieve



Comments:

Reviewed by: [redacted]

Md Nazri Mohidin, E.I.T
Experienced Laboratory Technician

ATTERBERG LIMITS
(ASTM D4318)

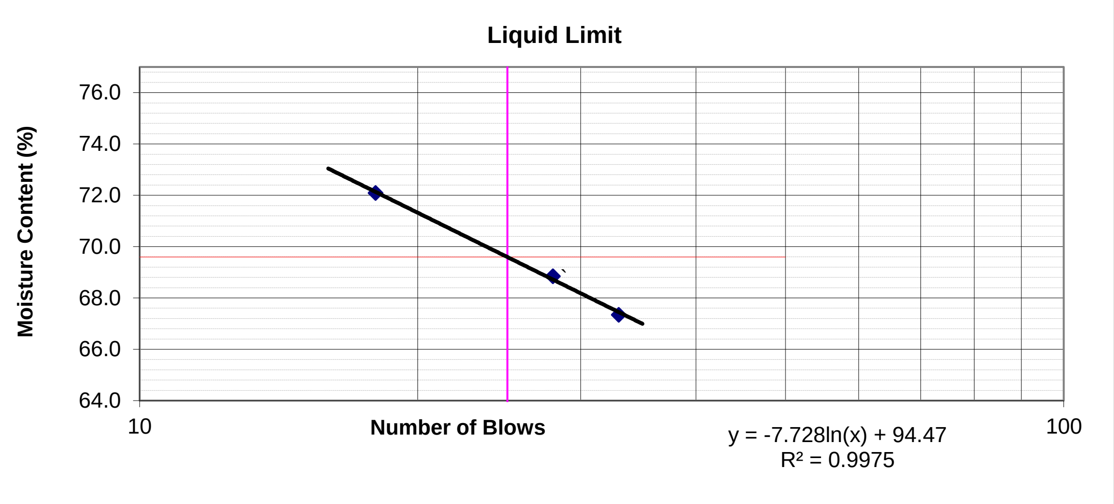
Client: WSP Canada Inc.
Project: Boissevain Lagoon
Site Location: Boissevain
Date Sampled: August 9, 2024
Sampled By: Lith Trinh

Lab No.: -
Project No.: CA0017114.3097
Report Date: July 17, 2024
Date Tested: July 16, 2024
Date Received: July 5, 2024

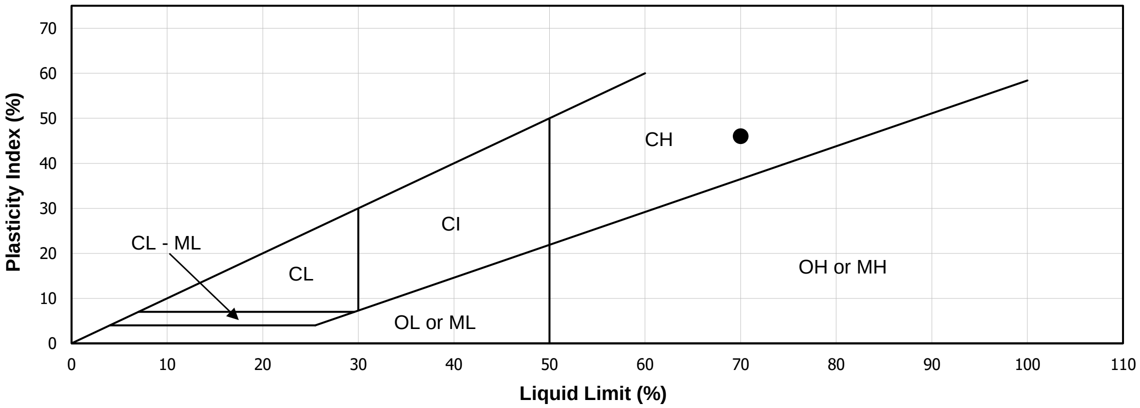
Testhole No.: TH24-12
Sample No.: S5
Depth (ft): 7.5
Drying Method: Oven Dried
Method: Multi-Point
Tested By: Rolando Rongcal

USCS Symbol:	CH
Plastic Limit (%):	24
Liquid Limit (%):	70
Plasticity Index:	**46**

	Liquid Limit			Plastic Limit	
	1	**2**	**3**	**1**	**2**
Wt. Of TARE	7.11	7.09	7.00	14.56	14.34
Wt. Of WET Sample and Tare	18.54	20.48	20.63	27.77	27.67
Wt. Of DRY Sample and Tare	13.94	15.02	14.92	25.18	25.07
Wt. Of WATER	4.60	5.46	5.71	2.59	2.60
Wt. Of DRY SAMPLE	6.83	7.93	7.92	10.62	10.73
PERCENT MOISTURE	**67.35**	**68.85**	**72.10**	**24.39**	**24.23**
Number of Blows	33	28	18		



Plasticity Chart for Soil Passing 425 um Sieve



Comments:

Reviewed by: [redacted]

Md Nazri Mohidin, E.I.T
Experienced Laboratory Technician

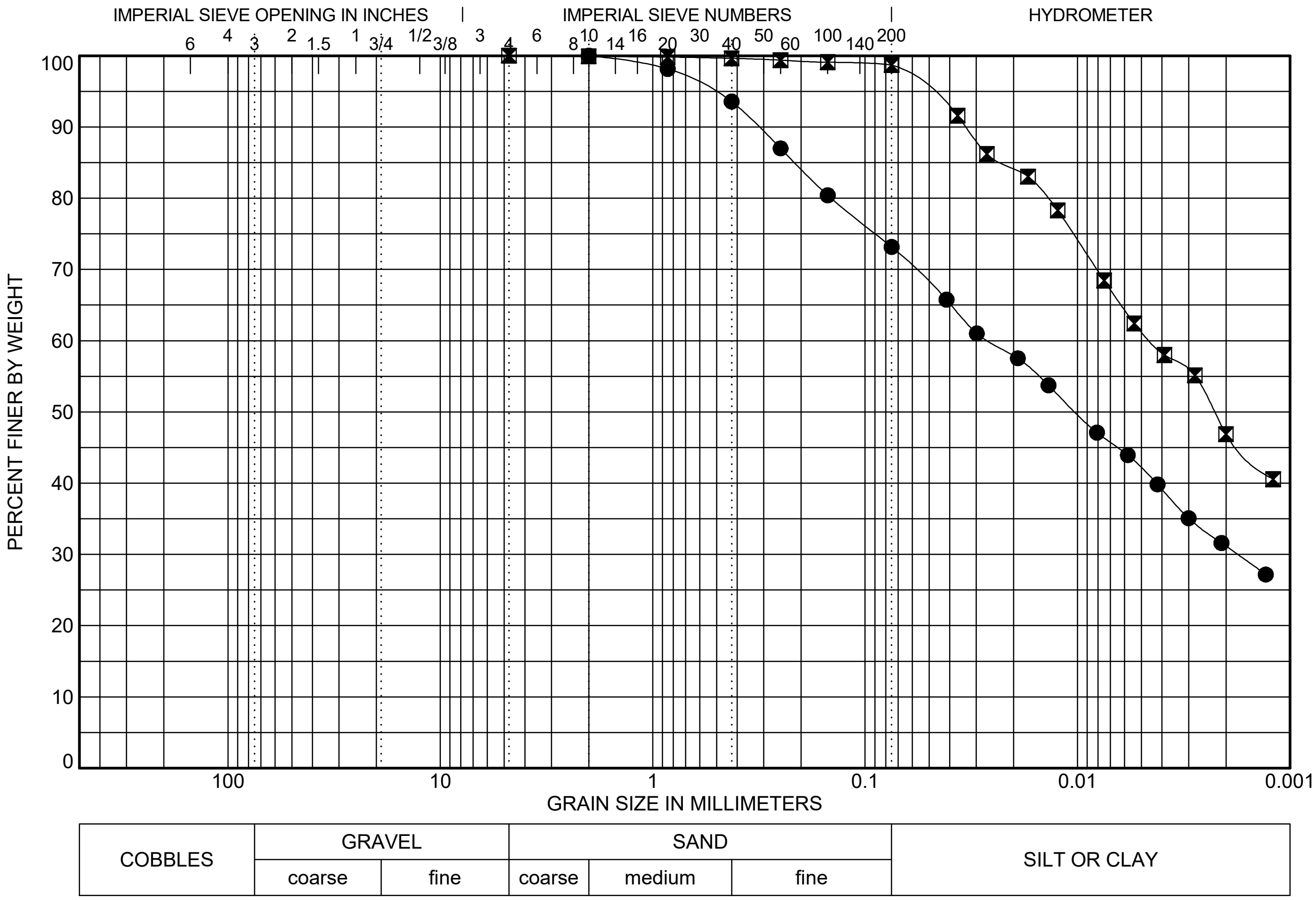
The test data given herein pertain to the sample provided. Reporting of these data constitutes a testing service. Engineering review and interpretation may be provided upon written request.

PARTICLE SIZE ANALYSIS

Report Date: 23 August 2024

Client		**Project**	
Name:	Manitoba Water Services Board	**Name:**	Boissevain Borrow Source
Address:	Unit 1A – 2010 Currie Blvd	**Address:**	NW-14/NW-13
Attention:	Nathan Wittmeier	**Project No.:**	CA0017817.1877
PO Number:		**Manager:**	NB

Gradation Specification:



	Sample ID	mUSCS	MC	D100	D60	D30	D10	Cc	Cu	% Gravel	% Sand	% Fines
●	TH24-09, 3 m	CH		2	0	0				0	27	42 (Silt) : 31 (Clay)
☒	TH24-12, 2.1 m	CH		4.8	0					0	1	52 (Silt) : 47 (Clay)

CA0017817.1877 - BOISSEVAIN BORROW SOURCE.GPJ 24/08/23 04:21 PM (WOOD - PSA MULTI RESULT SUMMARY)

Moisture / Density Relationship

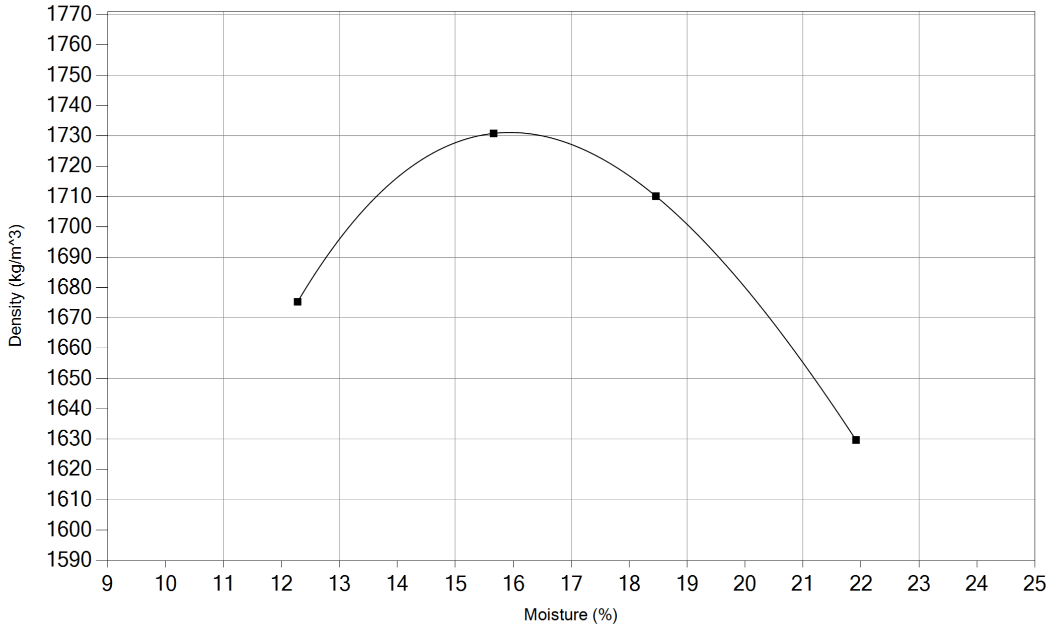
Report Date: August 23, 2024

Client

Name: WSP Canada Inc.
Address: 1600 Buffalo Place Winnipeg, MB R3T 6B8
Attention: Dana Bredin
PO Number:
Sample Date: 8/9/2024 by Lihn Trinh
Source: TH24-09
Sample S3 To S7

Project

Name: (CA0017817.1877) Boissevain Borrow Source
Address: Boissevain, MB
Phase: N/A **Task:** N/A
Manager: Randell Johnson
Lab/Ref. #: CA0017114.3097- Perm1
Description: Clay 1



Moisture Density Relationship: (ASTM D698-12) Method: A

Preparation Method: Dry **Rammer Type:**Mechanical

Maximum Density (kg/m^3): 1731

Optimum Moisture (%): 15.9

Remarks:

Distribution: Jason Bunn, Linh Trinh, Brianna Landrie, Nathan Boenders, Dana Bredin

Reviewed By: Md Nazri Mohidin

[redacted]

WSP Canada Inc.. - 6 High Level Road - Oak Bluff (Wpg), Mb

CCIL Certified Aggregate Type C, Type D & Concrete Aggregate Type R in accordance with CSA A283-19

ASTM D5084 - HYDRAULIC CONDUCTIVITY REPORT

TO:	Manitoba Water Services Board Unit 1A – 2010 Currie Blvd Brandon, Manitoba R7B 4E7	**PROJECT NO:**	CA0017817.1877
		CLIENT:	Manitoba Water Services Board
		DATE SUBMITTED:	
PROJECT:	Boissevain Borrow Source		

SAMPLE:	Remoulded Clay	**PERMEANT:**	De-Aired Tap Water
SAMPLE NO.:	Combined TH24-09	**HYDRAULIC GRADIENT:**	26.97
SAMPLE DEPTH:	1-11 ft		

CONSTANT HEAD METHOD (K = cQL/thA)

	Sample Height, L (cm)	Sample Diameter (cm)	Water Content (%)	Dry Density (kg/m^3)	Degree of Saturation (%)	Cell Pressure (kPa)	Back Pressure (kPa)	Differential Pressure, h (kPa)
Initial	7.89	7.30	16.0%	1683	71.4%	241.4	196.5	20.7
Final	7.96	7.39	27.6%	1549	100.1%			

Date & Time		Time, t	Flow (Q)		Temp.	Hyd. Cond.
Start	End	(seconds)	Influent (ml)	Effluent (ml)	Corr, c	Corrected, K (cm/s)
8/29/24 8:06 AM	8/29/24 4:50 PM	31440	3.00	0.75	1.213	6.47E-08
8/29/24 4:50 PM	8/30/24 7:52 AM	54120	5.15	5.25	0.931	8.00E-08
8/30/24 7:52 AM	8/30/24 3:20 PM	26880	2.85	2.80	0.931	8.76E-08
8/30/24 3:20 PM	9/3/24 8:20 AM	320400	23.80	23.10	0.931	6.10E-08
9/3/24 8:20 AM	9/4/24 9:20 AM	90000	4.80	4.40	0.931	4.26E-08
9/4/24 9:20 AM	9/4/24 5:45 PM	30300	2.50	2.60	0.956	7.20E-08
9/4/24 5:45 PM	9/5/24 8:30 AM	53100	3.75	3.20	0.980	5.74E-08
9/5/24 8:30 AM	9/6/24 8:50 AM	87600	5.45	5.55	0.980	5.51E-08
9/6/24 8:50 AM	9/6/24 1:50 PM	18000	1.10	1.05	0.980	5.24E-08

Comments:

Remoulded sample was prepared at 95% Maximum Dry Density.

Average Temperature Corrected Value (cm/s): 5.92E-08

Reviewed By: [redacted]

Md Nazri Mohidin, E.I.T
Experienced Laboratory Technician

Reporting of these results constitutes a testing service only.
Engineering interpretation or evaluation of the test results is provided only on written request.

Moisture / Density Relationship

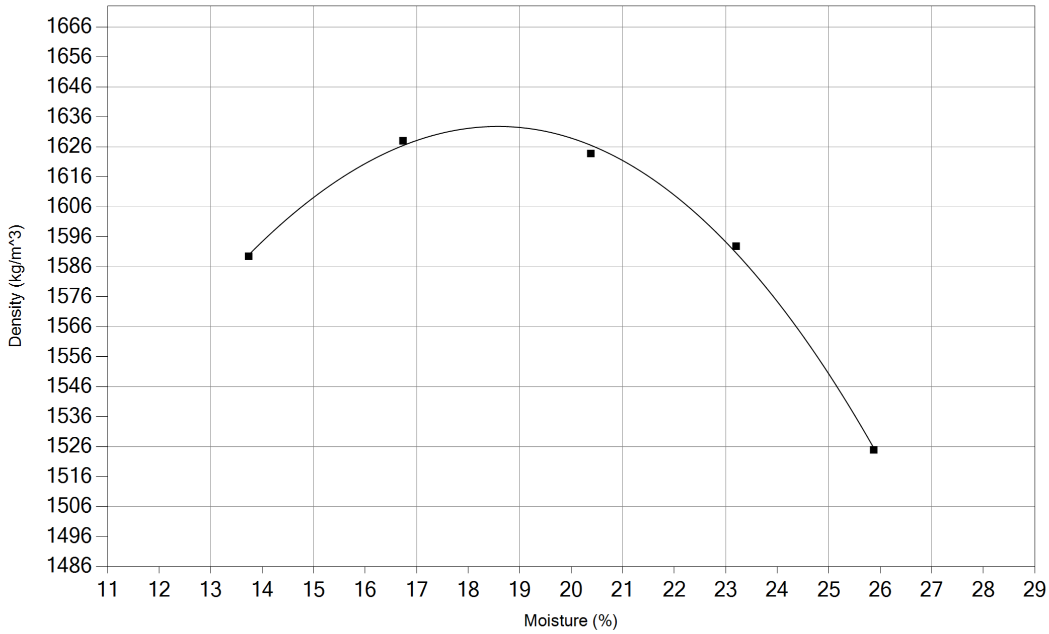
Report Date: August 23, 2024

Client

Name: WSP Canada Inc.
Address: 1600 Buffalo Place Winnipeg, MB R3T 6B8
Attention: Dana Bredin
PO Number:
Sample Date: 8/9/2024 by Lihn Trinh
Source: TH24-10 And TH24-12 Combined

Project

Name: (CA0017817.1877) Boissevain Borrow Source
Address: Boissevain, MB
Phase: N/A **Task:** N/A
Manager: Randell Johnson
Lab/Ref. #: CA0017114.3097- Perm2
Description: Clay 2



<u>Moisture Density Relationship:</u> (ASTM D698-12) Method: A

Preparation Method: Dry **Rammer Type:**Mechanical

Maximum Density (kg/m^3): 1633

Optimum Moisture (%): 18.6

Remarks:

Distribution: Jason Bunn, Linh Trinh, Brianna Landrie, Nathan Boenders, Dana Bredin

Reviewed By: Md Nazri Mohidin

[redacted]

Reporting of these results constitutes a testing service only. Engineering interpretation or evaluation of the test results is provided only on written request.

WSP Canada Inc.. - 6 High Level Road - Oak Bluff (Wpg), Mb

CCIL Certified Aggregate Type C, Type D & Concrete Aggregate Type R in accordance with CSA A283-19

ASTM D5084 - HYDRAULIC CONDUCTIVITY REPORT

TO: Manitoba Water Services Board
Unit 1A – 2010 Currie Blvd
Brandon, Manitoba
R7B 4E7

PROJECT NO: CA0017817.1877
CLIENT: Manitoba Water Services Board
DATE SUBMITTED:

PROJECT: Boissevain Borrow Source

SAMPLE: Remoulded Clay
SAMPLE NO.: Combined TH24-10 and TH24-12
SAMPLE DEPTH: 4-10 ft

PERMEANT: De-Aired Tap Water
HYDRAULIC GRADIENT: 26.97

CONSTANT HEAD METHOD (K = cQL/thA)

	Sample Height, L (cm)	Sample Diameter (cm)	Water Content (%)	Dry Density (kg/m^3)	Degree of Saturation (%)	Cell Pressure (kPa)	Back Pressure (kPa)	Differential Pressure, h (kPa)
Initial	7.82	7.32	18.2%	1627	74.5%	241.4	196.5	20.7
Final	7.90	7.36	23.6%	1675	104.2%			

Date & Time		Time, t (seconds)	Flow (Q)		Temp. Corr, c	Hyd. Cond. Corrected, K (cm/s)
Start	End		Influent (ml)	Effluent (ml)		
8/29/24 8:06 AM	8/29/24 4:50 PM	31440	3.10	1.60	1.213	8.00E-08
8/29/24 4:50 PM	8/30/24 7:40 AM	53400	4.40	6.30	0.931	8.23E-08
8/30/24 7:40 AM	8/30/24 5:20 PM	34800	3.00	2.40	0.931	6.37E-08
8/30/24 5:20 PM	9/3/24 8:20 AM	313200	27.00	21.60	0.931	6.37E-08
9/3/24 8:20 AM	9/4/24 9:20 AM	90000	7.80	7.20	0.931	6.84E-08
9/4/24 9:20 AM	9/4/24 5:45 PM	30300	1.65	1.50	0.956	4.38E-08
9/4/24 5:45 PM	9/5/24 8:30 AM	53100	3.20	3.20	0.980	5.21E-08
9/5/24 8:30 AM	9/5/24 5:10 PM	31200	1.70	1.80	0.980	4.85E-08
9/5/24 5:10 PM	9/6/24 8:00 AM	53400	3.10	3.20	0.980	5.10E-08
9/6/24 8:00 AM	9/6/24 3:45 PM	27900	1.20	1.30	0.980	3.87E-08

Comments:

Remoulded sample was prepared at 95% Maximum Dry Density.

Average Temperature Corrected Value (cm/s): 4.76E-08

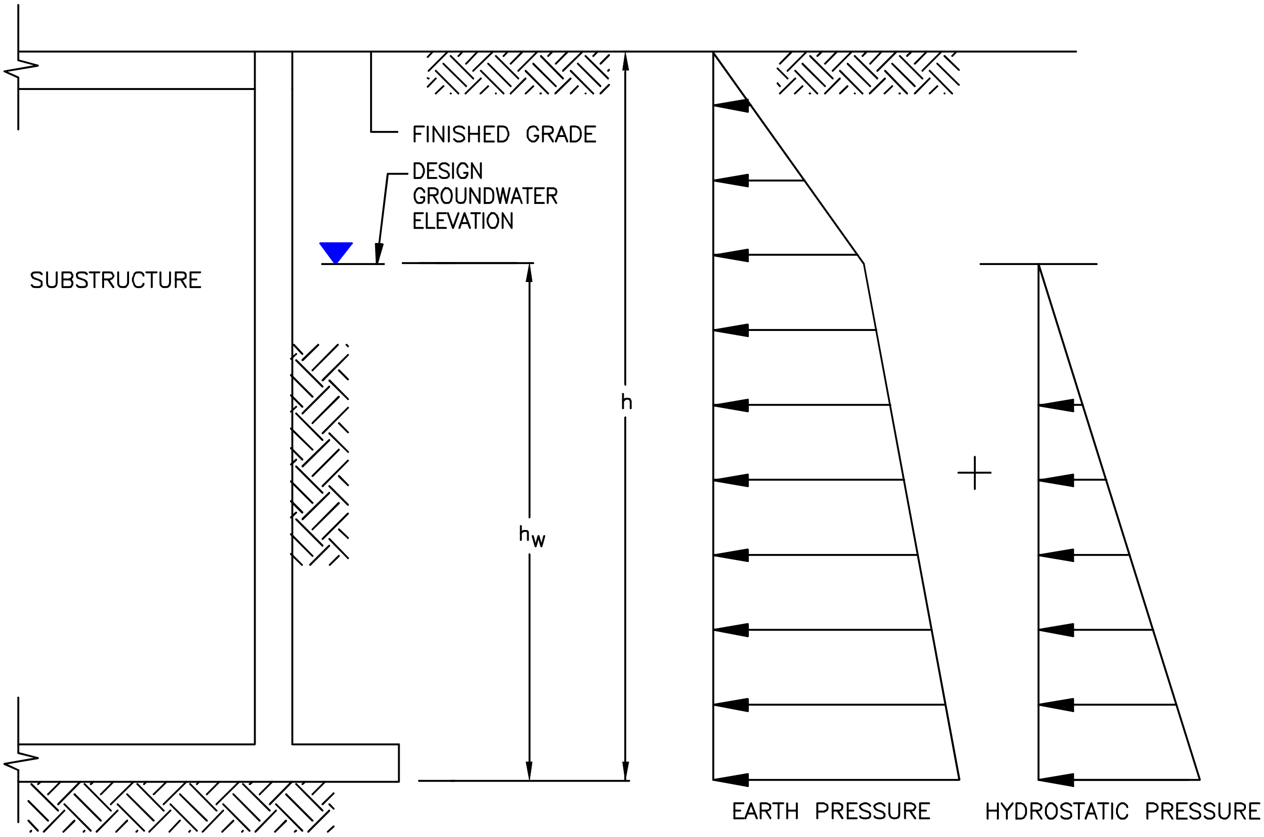
Reviewed By: [redacted]
Md Nazri Mohidin, E.I.T
Experienced Laboratory Technician

WSP E *Canada Limited*
6 High Level Road
Oak Bluff, Manitoba
R4G 0E2, Canada

APPENDIX

D

EARTH PRESSURE DIAGRAMS



RECOMMENDED PRESSURE DISTRIBUTION

$$P_h = K\gamma(h-h_w) + K\gamma' h_w + \gamma_w h_w$$

WHERE: P_h = LATERAL EARTH PRESSURE (kPa)
γ = UNIT WEIGHT OF SOIL (SEE TEXT OF REPORT)
γ' = BUOYANT UNIT WEIGHT OF SOIL (SEE TEXT OF REPORT)
γ_w = UNIT WEIGHT OF WATER (USE γ_w=9.8 kN/m^3)
h = HEIGHT OF WALL FROM FINISHED GRADE TO BASE OF WALL (m)
h_w = HEIGHT OF GROUNDWATER TABLE ABOVE BASE OF WALL (m)
K = LATERAL EARTH PRESSURE COEFFICIENT (K_a OR K_o – SEE REPORT TEXT)

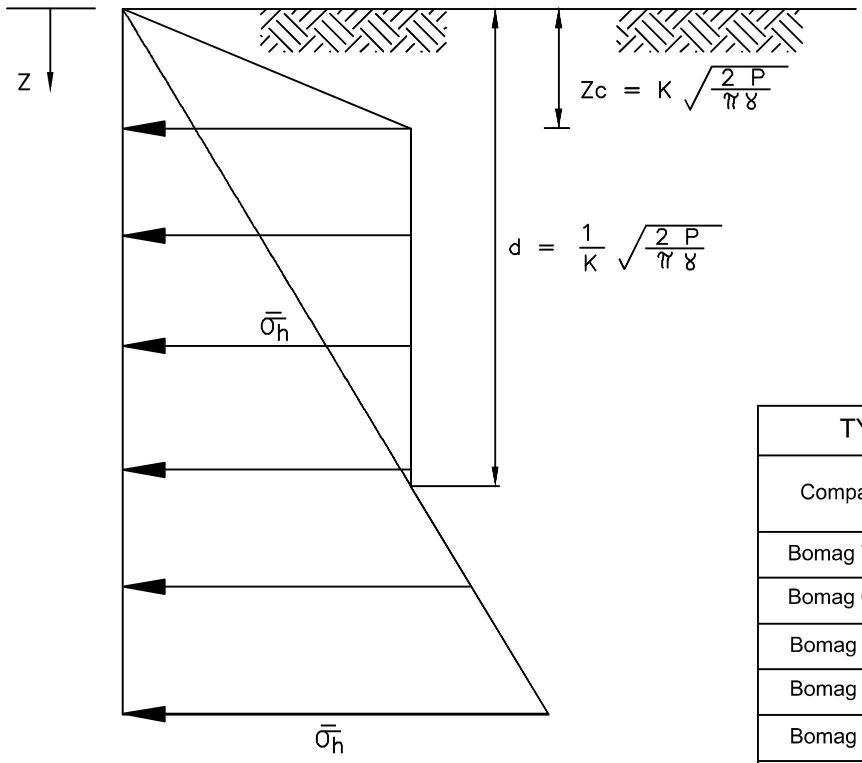
NOTES:

1. SEE TEXT OF REPORT FOR UNIT WEIGHTS AND LATERAL EARTH PRESSURE COEFFICIENTS.
2. A HYDROSTATIC COMPONENT NEED ONLY BE INCLUDED WHERE A PERCHED GROUND WATER TABLE WERE TO OCCUR WITHIN THE LEVEL OF SUB–STRUCTURE IN COMBINATION WITH NO SUB–DRAINAGE.

DWN BY: MD
CHK'D BY: RB
DATUM: -
PROJECTION: -
SCALE: NOT TO SCALE

LATERAL EARTH PRESSURE DISTRIBUTION
FOR LIGHTLY COMPACTED BACKFILL

DATE:
PROJECT No.:
REV. No.: A
FIGURE No.: FIGURE 1



EARTH PRESSURE DISTRIBUTION

FOR $Zc \leq Z \leq d$

$$\bar{\sigma}_h = \sqrt{\frac{2P\gamma}{\pi}}$$

FOR $Z > d$

$$\bar{\sigma}_h = K\cdot\gamma\cdot Z$$

TYPICAL COMPACTOR LOADS (P)			
Compactor	LOAD (P) kN/m	Compactor	LOAD (P) kN/m
Bomag TSE	31	Bomag BW122PD	36
Bomag 60S	32	Bomag 142PDB	47
Bomag 65S	22	Bomag 172PDB	93
Bomag 75S	33	Dynapac LR100	42
Bomag 90S	39	Dynapac 2100V	93
Bomag 75AD	20	Dynapac CA121D	53
Bomag 100AD	20	Dynapac CA121PD	54
Bomag 120AD	34	Dynapac CA151	80
Bomag 130AD	36	Dynapac CA151D	80
Bomag BW122D	30	Dynapac CA151PD	96

$$P \text{ (ROLLER LOAD)} = \frac{\text{DEAD WT. OF ROLLER + CENTRIFUGAL FORCE}}{\text{WIDTH OF ROLLER}}$$

TYPICAL VALUES GIVEN IN TABLE

EARTH PRESSURE COEFFICIENTS

$\bar{\sigma}_h$ = HORIZONTAL EARTH PRESSURE AT DEPTH Z

$K = K_o$ ("AT REST") OR K_a (ACTIVE CASE)
(SEE TEXT OF REPORT)

γ = SOIL UNIT WEIGHT
(SEE TEXT OF REPORT)

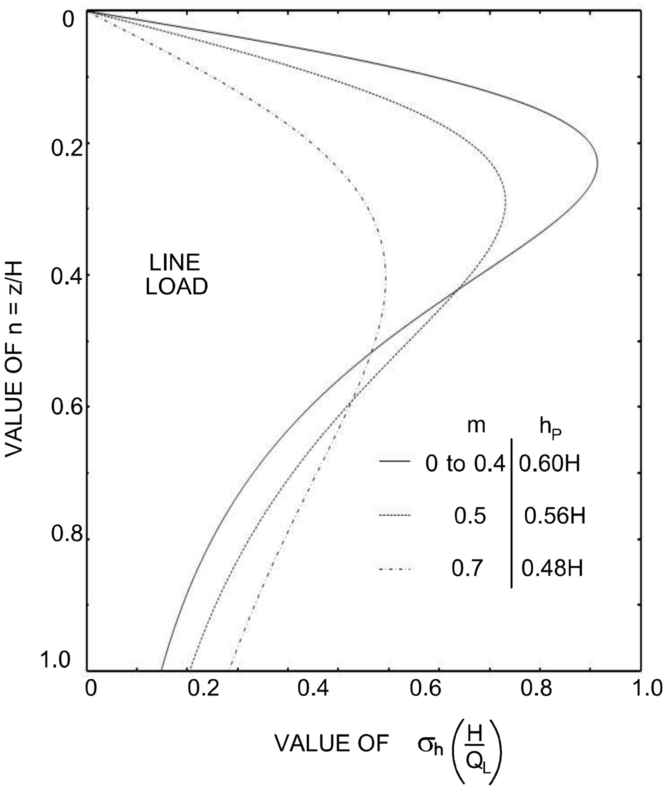
REFERENCE: INGOLD (1980), INTERNATIONAL CONFERENCE ON COMPACTION

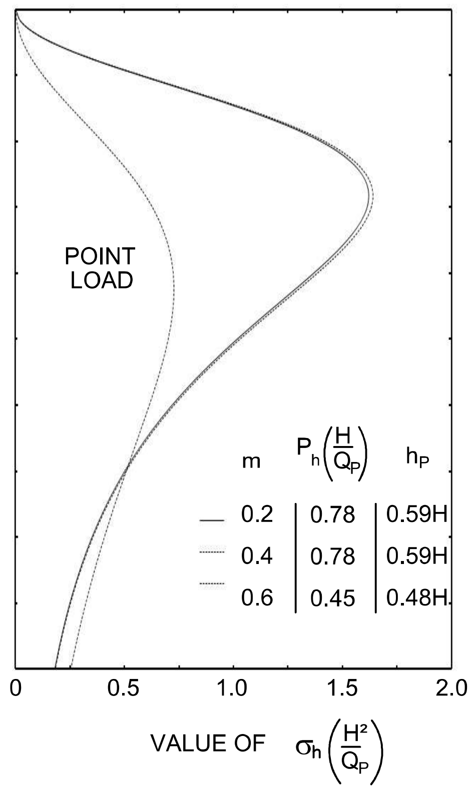
wsp

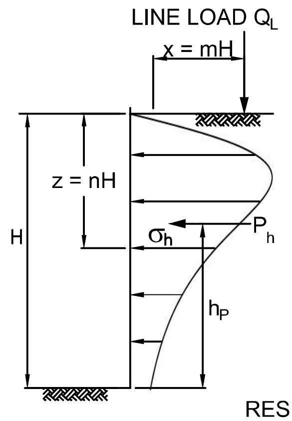
DWN BY: MD
CHK'D BY: RB
DATUM: -
PROJECTION: -
SCALE: NOT TO SCALE

LATERAL EARTH PRESSURES
INDUCED BY COMPACTION

DATE:
PROJECT No.:
REV. No.: A
FIGURE No.: FIGURE 2







FOR m ≤ 0.4:

$$\sigma_h\left(\frac{H}{Q_L}\right) = \frac{0.20n}{(0.16 + n^2)^2}$$

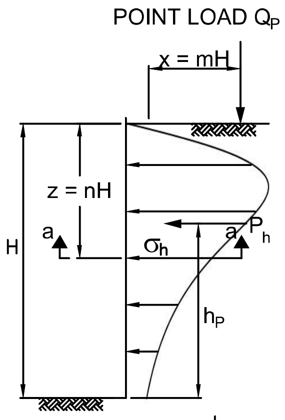
$P_h = 0.55\ Q_L$

FOR m > 0.4:

$$\sigma_h\left(\frac{H}{Q_L}\right) = \frac{1.28m^2n}{(m^2 + n^2)^2}$$

$$\text{RESULTANT } P_h = \frac{0.64\ Q_L}{(m^2 + 1)}$$

PRESSURES FROM LINE LOAD
(BOUSSINESQ EQUATION MODIFIED BY EXPERIMENT)



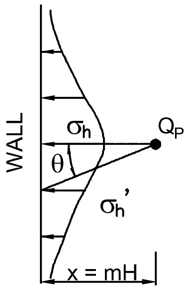
FOR m ≤ 0.4:

$$\sigma_h\left(\frac{H^2}{Q_P}\right) = \frac{0.28n^2}{(0.16 + n^2)^3}$$

FOR m > 0.4:

$$\sigma_h\left(\frac{H^2}{Q_P}\right) = \frac{1.77m^2n^2}{(m^2 + n^2)^3}$$

$$\sigma_h' = \sigma_h\ \text{COS}^2\ (1.1\theta)$$



SECTION a-a
PRESSURES FROM POINT LOAD
(BOUSSINESQ EQUATION
MODIFIED BY EXPERIMENT)

DWN BY: MD
CHK'D BY: RB
DATUM: -
PROJECTION: -
SCALE: NOT TO SCALE

LATERAL PRESSURES DUE TO
SURCHARGE POINT AND LINE LOADS

DATE:
PROJECT No.:
REV. No.: A
FIGURE No.: FIGURE 3