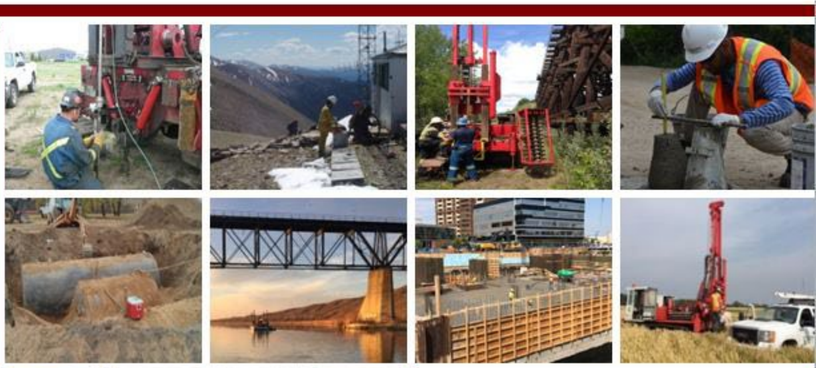
GEOTECHNICAL INVESTIGATION

PROPOSED POLE BUILDING
NORTH BATTLEFORD LANDFILL
NORTH BATTLEFORD, SASKATCHEWAN
PMEL FILE NO. 23764
MAY 13, 2026



PREPARED FOR:
City of North Battleford

ATTENTION: Kevin Kristian

PROJECT:

Geotechnical Investigation
Proposed Pole Building
North Battleford Landfill
North Battleford, Saskatchewan
PMEL File No. 23764
May 13, 2026

PREPARED FOR:

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1 INTRODUCTION

The following report has been prepared on the subsurface soil conditions existing at the site of the proposed pole building to be constructed at the North Battleford Landfill located in North Battleford, Saskatchewan. A Site Plan, showing the location of the study area and the boreholes has been shown on Drawing No. 23764-1.

The terms of reference for this investigation were presented in P. Machibroda Engineering Ltd. (PMEL) Proposal No. 293-342, dated March 31, 2026. Authorization to proceed with this investigation was provided in the signed Consulting Agreement between the City of North Battleford and PMEL, dated March 31, 2026.

2 FIELD INVESTIGATION

2.1 FIELD DRILLING PROGRAM

The field investigation was conducted on April 10, 2026.

Three boreholes, located as shown on the Site Plan, Drawing No. 23764-1, were dry drilled using our truck-mounted, continuous flight auger drilling rig. The boreholes were 150 mm in diameter and were extended to depths of 10.5 to 10.9 m below the existing ground surface. Borehole logs, as shown on the attached Drawing Nos. 23764-2 to 4, inclusive, were compiled during test drilling to record the soil stratification, the groundwater conditions, the position of unstable sloughing soils and the depths at which cobblestones and/or boulders were encountered.

Disturbed samples of auger cuttings, collected during test drilling, were sealed in plastic bags to minimize moisture loss. The soil samples were taken to our laboratory for analysis.

Standard penetration tests (SPT), utilizing a safety hammer with automatic trip were performed during test drilling.

2.2 FIELD SURVEY

The plan location and ground surface elevation of the boreholes was established using our handheld Global Positioning Equipment (Trimble, Model No. TDC6). The accuracy of the equipment is in the order of ± 0.5 m (depending on network signal strength).

3 SOIL AND GROUNDWATER CONDITIONS

3.1 SOIL PROFILE

The general soil profile consisted of a layer of recycled asphalt (0.7 to 0.9 m) at ground surface followed by fill that extended to depths of about 2.6 to 3.3 m below existing grade. Glacial till (clay till) was encountered beneath the fill and extended to the maximum depth explored during the field investigation (i.e., 10.9 m below existing ground surface). A layer of sand glacial till (sand till) was encountered between 5 and 6.3 m below existing ground surface at the location of BH 26-3.

Groundwater seepage and sloughing conditions were encountered in the sand till and intertill sand/gravel seam located in BH26-3. Thin layers of silt and clay were encountered between the fill and naturally occurring glacial till at the location of BH's 26-2 and 26-3.

The fill was comprised primarily of medium plastic glacial till but was intermixed with deposits of recycled asphalt pieces, clay, silt and organics. Relatively high SPT blow counts were encountered in two boreholes at 1.5 m below existing ground surface (i.e., N index = 24) suggesting the fill had a high strength at this depth. The clay/silt layers were firm and low to medium plastic. The underlying clay till was low to medium plastic and predominantly hard in consistency. The sand till layer was well graded, fine to coarse grained and dense.

3.2 GROUNDWATER CONDITIONS, SLOUGHING

Groundwater seepage and trace sloughing conditions were encountered in the sand deposits encountered in BH 26-3 during test drilling. The depths at which groundwater seepage and sloughing were encountered have been shown plotted on Drawing Nos. 23764-2 to 4, inclusive. Higher groundwater conditions could be encountered, particularly during/following spring thaw and/or periods of precipitation.

3.3 COBBLESTONES AND BOULDERS

Glacial till consists of a heterogeneous mixture of gravel, sand, silt and clay sized particles. Glacial till inherently contains sorted deposits of the above particle sizes as well as a random distribution of larger particle sizes in the cobblestone range (60 to 200 mm) and boulder-sized range (larger than 200 mm). Inter/intra till deposits of cobblestones, boulders, boulder pavements and isolated deposits of saturated sand or gravel should be expected.

It should be recognized that the statistical probability of encountering cobbles/boulders in the small diameter boreholes drilled at this site was low. The frequency of encountering such deposits will increase proportionately with the number/depth of piles installed and/or volume of soil excavated.

4 LABORATORY ANALYSIS

The soil classification and index tests performed during this investigation consisted of a visual classification of the soil, moisture contents, Atterberg limits, unit weights and grain size distribution analysis.

The results of the soil classification and index tests conducted on representative samples of soil have been plotted on the borehole logs alongside the corresponding depths at which the samples were recovered, as shown on Drawing Nos. 23764-2 to 4, inclusive. The results of the grain size distribution analysis have been presented in Appendix B.

5 DESIGN RECOMMENDATIONS

Based on the foregoing outline of soil test results, the following foundation considerations and design recommendations have been presented.

5.1 DESIGN CONSIDERATIONS

It is understood that a proposed pole building will be constructed and recommendations for suitable foundation types are required. It is also understood that the building will be heated and the existing recycled asphalt covered ground surface will serve as the floor for the building.

The general soil profile consisted of a surface layer of recycled asphalt (0.7 to 0.9 m) followed by fill to depths of about 2.6 to 3.3 m then glacial till to the maximum depth explored during the field investigation (i.e., 10.5 m below existing ground surface). Groundwater seepage and sloughing conditions were encountered in sand/gravel seams/layers at the location of BH 26-3. Higher groundwater conditions could be encountered, particularly during/following spring thaw and/or periods of precipitation.

The subgrade soils are considered frost susceptible, and the potential depth of frost penetration could range from approximately 1.8 to 2.6 m, depending on surface cover, severity of winter and heat loss affects beneath/adjacent buildings.

Drilled, cast-in-place concrete piles should perform satisfactorily as a foundation support for the proposed building. Groundwater seepage/sloughing conditions were encountered in the glacial till during test drilling. Temporary casing will likely be required.

Helical screw piles could also be considered but may encounter construction difficulties due to hard ground conditions and potential for encountering cobbles/boulders which could lead to high installation torques and premature pile termination. Pre-drilling and/or supplemental piles could be required if shallow termination is encountered.

Footings were considered as a foundation support but are not recommended due to the extensive thickness of heterogeneous fill encountered at the site. Footings based on the fill could undergo an unacceptable level of performance associated with uneven foundation settlement.

Recommendations have been prepared for site preparation; limit states resistance factors and serviceability; deep foundations; foundation concrete and site classification for seismic site response.

5.2 LIMIT STATES RESISTANCE FACTORS AND SERVICEABILITY

The National Building Code of Canada (NBCC, 2020) requires the use of limit states design for the design of buildings and their structural components, including the design of shallow and deep foundations. It is expected that the designer is familiar with the limit states design method and only a brief discussion will be presented. For a detailed discussion, it is recommended to review the NBCC (2020) and/or the Canadian Foundation Engineering Manual (CFEM, 2023).

Limit states are defined as those conditions under which a structure ceases to fulfill the function for which it was designed (i.e., unsatisfactory performance). In limit states design, two conditions are assessed with respect to performance, these are:

- ultimate limit states (ULS), and
- serviceability limit states (SLS)

Ultimate limit states are concerned with the collapse mechanisms of the structure (i.e., safety), whereas serviceability limit states consider mechanisms that restrict or constrain the intended use, function or occupancy of the structure.

As per NBCC (2020), the factored soil resistance utilized for foundation design may be determined using the following resistance factors applied to the ultimate resistance values presented in the following subsections of the report.

Deep foundations:

- Compressive Resistance, $\Phi = 0.4$
- Tensile Resistance, $\Phi = 0.3$

The above resistance factors have been provided to reflect that semi-empirical methods were used to derive the soil bearing resistances presented in this report using the laboratory and in-situ data collected during this investigation.

To satisfy serviceability limit states, a settlement analysis of the foundation must also be evaluated to ensure the structures are not negatively impacted by excessive settlement at the design load. Estimated foundation settlements have been provided in Section 5.3.3.

Piles exposed to lateral loads are typically designed to restrict lateral deflection of the pile head to tolerable limits. Lateral pile head deflection can be determined using the concepts presented in Section 5.3.4.

5.3 DEEP FOUNDATIONS

5.3.1 DRILLED, CAST-IN-PLACE CONCRETE PILES

Drilled, cast-in-place, straight shaft concrete piles should be designed on the basis of shaft resistance only. The ULS and SLS resistance values for design of drilled piles have been presented in Table I.

TABLE I SHAFT RESISTANCE (DRILLED PILES)

Depth (m)[1]	Shaft Resistance (kPa)	
	Unfactored ULS	SLS
0 to 3	0	0
3 to 3.5	50	20
3.5 to 5	80	32
Below 5	110	44

Notes:

1. For the purposes of this report, design depths have been referenced to existing grade. The structural engineer must consider finished grade elevation relative to existing grade. If existing grade is altered significantly, PMEL should be consulted to confirm the design parameters.
2. Minimum pile lengths should take into account the depth required to resist frost action (refer to Section 5.3.6).
3. Piles should be reinforced to withstand all axial and lateral forces within the pile.
4. A minimum pile diameter of 400 mm is recommended for the primary structural loads. Larger pile diameters may be required to allow for the removal of cobbles and boulders in some pile holes.
5. The pile holes should be filled with concrete as soon as practical after drilling.
6. Casing will be required where groundwater seepage and sloughing conditions are encountered to maintain the pile holes open for placing of the reinforcing steel and concrete. The annular space between the casing and drilled hole must be filled with concrete. As casing is extracted, concrete in casing must have adequate head to displace all water in the annular space.
7. A minimum centre-to-centre pile spacing of not less than three pile diameters is recommended.
8. A representative of the Geotechnical Consultant should inspect and document the installation of the drilled, cast-in-place concrete piles.

5.3.2 HELICAL SCREW PILES

Construction difficulties associated with strong ground conditions and potential cobbles/boulders should be expected during installation of helical screw piles. Pre-boring and/or installation of supplemental piles may be required.

Helical screw piles are installed by rotating a steel pipe, equipped with one or more helix flightings, into the ground. Due to the strong ground conditions, multi-helix screw piles are not recommended at the site.

Pile capacity may be derived using shearing resistance along the pile shaft (i.e., shaft resistance) as well as end bearing capacity of the helix.

The ULS and SLS soil resistance values for design of screw piles have been presented in Table II.

TABLE II SHAFT RESISTANCE (SCREW PILES)

Depth	Shaft Resistance (kPa)	
	Unfactored ULS	SLS
0 to 3	0	0
3 to 3.5	25	10
Below 3.5	40	16

TABLE III END BEARING RESISTANCE (SCREW PILES)

Depth (m) / Bearing Strata	*Helix Diameter (m)	End Bearing Resistance (kPa)			
		Compressive		Tensile	
		Unfactored ULS	SLS	Unfactored ULS	SLS
Below 4.5/Hard Glacial Till	≤ 0.5	1,800	600	1,125	375
	≥ 1.0	1,200	400	750	250

Screw piles must be based in hard glacial till, depth will vary across the site. Piles should extend at least 2 helix diameters into hard glacial till (or as far as possible within the limits of the structural torque capacity of the piles). Torque monitoring must be conducted and a representative of PMEL must be on-site to confirm piles achieve adequate penetration into the bearing soils.

* For helix sizes between 0.5 m and 1 m, end bearing resistance can be determined by linear interpolation between the ≤ 0.5 and ≥ 1.0 values.

Notes:

1. The helix should be embedded a minimum of 5D (D = helix diameter) below existing grade. Piles exposed to frost action may need to be longer and should be designed to resist frost jacking forces (refer to Section 5.3.6).

2. When determining the compressive resistance of the pile shaft, the portion of the pile shaft within 1D above the helix should be discounted due to interaction effects between the pile shaft and helix. For piles subject to tensile loads, the zone of zero shaft resistance should be increased to 2D above the helix.

3. Compressive end bearing capacity may be calculated utilizing the effective soil contact area of the helix (i.e., overall cross-sectional area of the helix, including the central shaft). Piles subject to tensile loads should use the effective area of the helix (i.e., helix area minus shaft area) when determining uplift pile capacity.

4. A minimum centre-to-centre pile spacing of 2.5D is recommended. Lesser spacings may be acceptable but must be approved by the Geotechnical Consultant.

5. The helical plate shall be normal to the central shaft (within 3 degrees) over its entire length. Multiple helixes (if applicable) should be spaced at increments of the helix pitch to ensure that all helixes travel the same path during installation.

6. Continuous monitoring of the installation torque should be undertaken during installation to determine whether the screw pile has been damaged during installation and to monitor the consistency of the subsurface soils.

7. Screw piles should be designed on the basis of conventional static analysis using the resistance values presented above. Installation torque should be used for monitoring purposes only and not to determine pile capacity.

8. The installation of screw piles typically disturbs the upper portion of the soils, often resulting in poor to no contact with the adjacent soils in this zone. As such, additional measures may be required if screw piles are required to resist lateral loading (i.e., pre-boring and backfilling of the annular space with lean mix concrete, construction of a buried pile cap/grade beam over the screw pile, use of larger diameter pile shafts etc.). If screw piles are required to resist lateral loads, the design details should be reviewed with the Geotechnical Consultant.

9. A representative of the Geotechnical Consultant should inspect and document the installation of each screw pile on a continuous basis.

5.3.3 PILE SETTLEMENT

With regards to serviceability of pile foundations, assuming good construction practices are followed and the appropriate resistance factors are applied; the settlement of individual piles at the design load will be small and should be within tolerable limits. The estimated pile settlement at working loads should be in the order of 5 to 10 mm for drilled, straight shaft concrete piles and 10 to 20 mm for helical screw piles.

The above is applicable to individual piles and small pile groups. Although not anticipated, foundation settlement should be evaluated where large pile groups are employed to carry the foundation load (i.e., breadth of foundation or pile cap is a similar dimension as depth of piles).

Pile foundations designed utilizing the provided SLS bearing capacities would perform similarly to pile foundations designed using the provided ULS capacities.

5.3.4 LATERAL THRUST FORCES

Pile deflection typically governs the design of laterally loaded piles. Subgrade reaction theory may be utilized to estimate lateral pile deflection. The estimated coefficients of horizontal subgrade reaction of the subgrade soils have been presented in Table IV.

TABLE IV ESTIMATED COEFFICIENTS OF HORIZONTAL SUBGRADE REACTION

Zone (m) [1]	Coefficient of Horizontal Subgrade Reaction, Ks, (kN/m^3)
0 to 1.5D	0
1.5D to 3.5	7,000/D
3.5 to 5	20,000/D
Below 5	30,000/D

[1] Depth below existing ground level.

Where D = pile diameter (m). For large diameter piles (i.e. exceeding 1.0 m) the zone of zero horizontal subgrade reaction should not exceed 1.5 m.

For the purposes of this report, design depths have been referenced to existing grade. The structural engineer must consider finished grade elevation relative to existing grade. If existing grade is altered significantly, PMEL should be consulted to confirm the design parameters.

The response of a pile to lateral loads is highly nonlinear. Methods that assume linear behaviour, such as horizontal subgrade reaction theory, are only applicable where pile deflections are small, loading is static and pile materials are linear; these conditions do not exist in most cases and soil-pile interaction modeling (i.e., p-y method) is required to accurately model the pile behaviour. If a more detailed lateral analysis is deemed warranted, PMEL can model the interaction between the soil and the pile, in accordance with the p-y method. Specific pile details (i.e., loading, type, diameter, length, etc.) will be required in order to perform the analysis.

The installation of screw piles typically disturbs the upper portion of the soils, often resulting in poor to no contact with the adjacent soils in this zone. As such, additional measures may be required if screw piles are required to resist lateral loading (i.e., pre-boring and backfilling of the annular space with lean mix concrete, construction of a buried pile cap/grade beam over the screw pile, etc). If screw piles are required to resist lateral loads, the design details should be reviewed with the Geotechnical Consultant.

5.3.5 GRADE BEAMS AND PILE CAPS

Grade beams and pile caps should be reinforced at both top and bottom throughout their entire length/cross section. Grade beams (and pile caps exposed to frost action) should be constructed to allow for a minimum of 100 mm of net void space between the underside of the grade beam/pile cap and the subgrade soil (compressible void form). The finished grade/floor finish adjacent to all pile caps and grade beams should be such that water runoff is not allowed to infiltrate and collect in the void space.

5.3.6 FROST JACKING OF DEEP FOUNDATIONS

Frost jacking is a process that can cause progressive upward movement of piles due to adfreeze bond stresses (adfreeze) between the soil and the pile shaft within the depth of frost penetration. Frost jacking requires exposure to freezing conditions and frost-susceptible soils.

Silty, weak or wet soils and shallow groundwater conditions typically amplify the potential for and severity of frost jacking.

The subgrade soils are frost susceptible and the potential depth of frost penetration could range from about 1.8 m (lower bound) to 2.6 m (upper bound), depending on surface cover, severity of winter and heat loss effects beneath/adjacent to buildings.

Piles in unheated/intermittently heated areas (particularly those supporting negligible to light loads) are particularly susceptible to frost jacking and must be designed to resist frost jacking forces resulting from the upper bound frost penetration depth (2.6 m).

Interior piles below a heated space (i.e., installed during non-freezing conditions and installed below continually heated areas) will be unaffected by frost jacking.

Perimeter piles installed below continually heated areas will experience reduced frost jacking forces (as compared to unheated areas), provided that the building envelope is designed to allow heat loss to the foundation (i.e., where the floor slab is insulated, an uninsulated strip at least 1 m wide should be provided adjacent to the perimeter foundation). In this case, the perimeter piles should be designed to resist frost jacking forces resulting from the lower bound frost penetration depth (i.e., 1.8 m).

If heat loss to the foundation is not allowed (i.e., fully insulated building envelope), the perimeter piles should be designed to resist frost jacking forces due to the upper bound frost penetration depth (i.e., 2.6 m).

Adfreeze values are difficult to quantify accurately and can vary depending on many factors. For the purposes of this report, an adfreeze value of 100 kPa is recommended for concrete piles and 150 kPa for steel piles.

Piles subject to frost action can resist frost jacking in two ways:

1. Structural resistance due to pile self-weight plus sustained (unfactored) structural loading applied to the pile head; and,

2. Geotechnical resistance due to soil/pile interaction below the depth of frost penetration.

To determine the maximum frost jacking force, the structural designer should consider the maximum adfreeze value and the recommended design frost penetration depth, as discussed above. The frost jacking force that the pile should be designed to resist would be equal to the maximum frost jacking force minus the structural resistance (i.e., point 1 above).

To determine the geotechnical resistance of the pile to resist frost jacking (i.e., point 2 above), the structural designer should consider the unfactored ULS resistance values presented in the previous sections of this report (i.e., resistance factor of 1.0) applied to the soils below the recommended design frost penetration depth.

The potential for frost jacking can be reduced through prudent design and good construction practices. Such measures may include:

- Provide adequate site drainage (overland and/or subsurface) to minimize water accumulation adjacent to foundations;
- Maintain uniform pile shaft cross sections; avoid enlarged/tapered pile tops which can increase the surface area for frost to act on;
- Reduce the potential depth of frost penetration by heating and/or insulating the area;
- Fill open steel piles with concrete to prevent water accumulation within the pile; and,
- Utilize bond breakers between the pile and the soil within the depth of frost penetration (e.g., Sonotube forms, polyethylene sleeves, plastic wrapping, low friction coatings etc.). It is noted that some bond breakers will not be suitable for piles subject to lateral loading due to a gap between the soil and the pile.

5.4 FOUNDATION CONCRETE

The results of water-soluble sulphate testing on previous projects conducted in North Battleford have shown that severe concentrations of sulphates exist in the subgrade soils in this geographic region. For this reason, it is recommended to utilize sulphate resistant cement (minimum S-2 class of exposure) for all foundation concrete in contact with the subgrade soils. All concrete at this site should be manufactured in accordance with current CSA standards.

It should be recognized that water soluble sulphate salts, combined with moist soils or low pH soils could render the soil highly corrosive to some types of metals in contact with the soil.

5.5 SITE CLASSIFICATION FOR SEISMIC SITE RESPONSE

Based on the consistency of the subgrade soils encountered at this site and Table 4.1.8.4B of the 2020 National Building Code, the site classification for seismic site response falls within Class D.

6 LIMITATIONS

The presentation of the summary of the borehole logs and design recommendations has been completed as authorized. Three, 150 mm diameter boreholes were drilled using our continuous flight, solid stem auger drilling equipment. Borehole logs were compiled during test drilling which, we believe, were representative of the subsurface conditions at the borehole locations at the time of test drilling.

Variations in the subsurface conditions from that shown on the borehole logs at locations other than the exact test locations should be anticipated. If conditions should differ from those reported here, then we should be notified immediately in order that we may examine the conditions in the field and reassess our recommendations in the light of any new findings.

The Terms of Reference for this investigation did not include any environmental assessment of the site. No detectable evidence of environmentally sensitive materials was detected during the actual time of the field test drilling program. If, on the basis of any knowledge, other than that formally communicated to us, there is reason to suspect that environmentally sensitive materials may exist, then additional boreholes should be drilled and samples recovered for chemical analysis.

The subsurface investigation necessitated the drilling of deep boreholes. The boreholes were backfilled at the completion of test drilling. Please be advised that some settlement of the backfill materials will occur which may leave a depression or an open hole. It is the responsibility of the client to inspect the site and backfill, as required, to ensure that the ground surface at each borehole location is maintained level with the existing grade.

This report has been prepared for the exclusive use of the City of North Battleford and their agents for specific application to the proposed pole building to be constructed at the North Battleford Landfill located in North Battleford, Saskatchewan. It has been prepared in accordance with generally accepted geotechnical engineering practices and reflects PMEL's understanding of the project based on information available at the time of preparation of this report. No other warranty, expressed or implied, is made.

The report should be referenced in its entirety, in order to properly understand the suggestions, design considerations and recommendations provided in this report. Any use which a Third Party makes of this report, or any reliance on decisions to be made based on it, is the responsibility of such Third Party. Governing Agencies such as municipal, provincial, or federal agencies having jurisdictions with respect to this development and/or construction of the facilities described herein have full jurisdiction with respect to the described development.

Any other unspecified subsequent development would be considered Third Party and would, therefore, require prior review by PMEL. PMEL accepts no responsibility for damages, if any, suffered by any Third Party as a result of decisions made or actions based on this report.

Prior to completion of the final design drawings/specifications, PMEL should be retained to review the geotechnical aspects of the project plans and documents to confirm that they are consistent with the intent of this report.

If this report has been transmitted electronically, it has been digitally signed and secured with personal passwords to lock the document. Due to the possibility of digital modification, only those reports sent directly by PMEL can be relied upon without fault.

7 LETTER OF COMMITMENT AND FIELD REVIEWS

The acceptance of responsibility for the design/construction recommendations presented in this report are contingent on PMEL providing field documentation and review services at the time of construction. Field reviews are necessary for PMEL to provide letters of assurance in accordance with requirements of local regulatory authorities. PMEL will not accept any responsibility on this project for any unsatisfactory performance if adequate and/or full-time inspection is not performed by a representative of PMEL.

The following field reviews are required for PMEL to complete the Letter of Commitment.

- ☒ Pile installation documentation (full-time)
- ☐ Shallow foundation subgrade review
- ☐ Review of soil compaction frequency and testing results
- ☐ Review of all proposed fill soils prior to placement of the fill
- ☐ Stability of temporary excavations
- ☐ Final site review following completion of landscaping/grading
- ☐ Other:

PMEL will rely upon the client and/or general contractor to notify PMEL on the progress of construction to allow PMEL to complete field reviews. Notice of when site is ready for field reviews to be completed must be provided in a timely manner. Additionally, PMEL must be notified immediately during construction if site conditions change or vary from the conditions provided in our geotechnical report. The owner, coordinating professional, and general contractor should all be made aware of the required field reviews. If sufficient/adequate field reviews are not completed during construction due to PMEL not being notified about construction progress, PMEL will be unable to provide a final signed Letter of Commitment for this project

8 CLOSURE

We trust that this report fulfills your requirements for this project. Should you require additional information, please contact us.

P. MACHIBRODA ENGINEERING LTD.

Association of Professional Engineers &
Geoscientists of Saskatchewan
CERTIFICATE OF AUTHORIZATION
P. MACHIBRODA ENGINEERING LTD.
Number 172
Permission to Consult held by:

Discipline	Sk. Reg. No.	Signature
Geotechnical	10461	K. Pardoski

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DRAWINGS

NOTE:
1. THIS DRAWING IS FOR CONCEPTUAL PURPOSES ONLY. ACTUAL LOCATIONS MAY VARY AND NOT ALL STRUCTURES ARE SHOWN.
2. THIS DRAWING WAS COMPILED FROM GOOGLE EARTH PRO ©2025, IMAGE ©2025 DIGITALGLOBE, (IMAGERY DATE: 05/27/24).
3. THIS DRAWING WAS COMPILED USING HANDHELD GPS EQUIPMENT (TRIMBLE TDC6, MODEL No. DA2).

LEGEND

⊕ –PMEL BOREHOLE

CONSULTING
GEOENVIRONMENTAL
GEOTECHNICAL
ENGINEERS

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DRAWING TITLE:
SITE PLAN - BOREHOLE LOCATIONS

PROJECT:
PROPOSED POLE BUILDING
NORTH BATTLEFORD LANDFILL, NORTH BATTLEFORD, SK

APPROVED BY: KP
DRAWN BY: TP
DRAWING NUMBER: 23764-1
DATE: APRIL, 2026
SCALE: AS SHOWN

P.MACHIBRODA ENGINEERING LTD.

BOREHOLE 26-1

DRAWING NUMBER: 23764-2

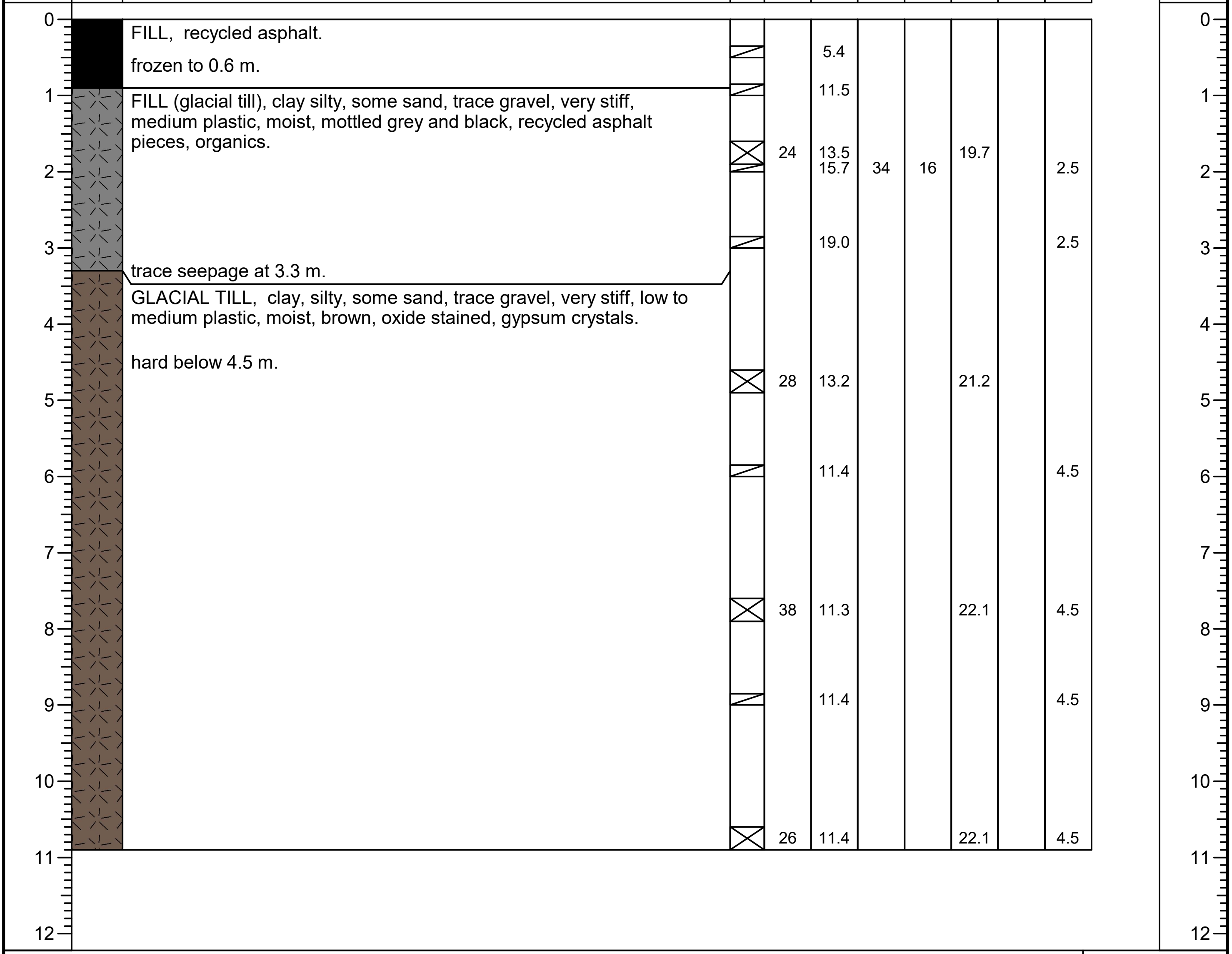
PROJECT: PROPOSED POLE BUILDING
NORTH BATTLEFORD LANDFILL

LOCATION: NORTH BATTLEFORD, SK

NORTHING (m): 5852349 **EASTING (m):** 685597 **ELEVATION (m):** 562.4 (+/-0.5 m) **DATE DRILLED:** APR 10/26

SAMPLE TYPE: ⧄ CUTTINGS ☒ SPLIT SPOON ■ SHELBY TUBE

WATER LEVELS
▼ After Drilling
▽ During Drilling



FILL, recycled asphalt.

frozen to 0.6 m.

FILL (glacial till), clay silty, some sand, trace gravel, very stiff, medium plastic, moist, mottled grey and black, recycled asphalt pieces, organics.

trace seepage at 3.3 m.

GLACIAL TILL, clay, silty, some sand, trace gravel, very stiff, low to medium plastic, moist, brown, oxide stained, gypsum crystals.

hard below 4.5 m.

SAMPLE TYPE	SPT (N) BLOWS/ 300 mm	WATER CONTENT (%)	LIQUID LIMIT (%)	PLASTIC LIMIT (%)	UNIT WEIGHT (kN/m³)	COMPRESSIVE STRENGTH (kPa)	POCKET PEN. (kg/cm²)
⧄		5.4					
⧄		11.5					
☒	24	13.5			19.7		
⧄		15.7	34	16			2.5
⧄		19.0					2.5
☒	28	13.2			21.2		
⧄		11.4					4.5
☒	38	11.3			22.1		4.5
⧄		11.4					4.5
☒	26	11.4			22.1		4.5

NOTES:
1. Borehole open and dry Immediately After Drilling.

SHEET 1 OF 1

05-13-2026 Y:\2026 Projects\23764 - Geo, Proposed Building, North Battleford Landfill, North Battleford\Drafting\Dwg\BH1-23764.bo

P.MACHIBRODA ENGINEERING LTD.

BOREHOLE 26-2

DRAWING NUMBER: 23764-3

PROJECT: PROPOSED POLE BUILDING
NORTH BATTLEFORD LANDFILL

LOCATION: NORTH BATTLEFORD, SK

NORTHING (m): 5852340 **EASTING (m):** 685589 **ELEVATION (m):** 562.2 (+/-0.5 m) **DATE DRILLED:** APR 10/26

SAMPLE TYPE: ⧄ CUTTINGS ☒ SPLIT SPOON ■ SHELBY TUBE

WATER LEVELS
▼ After Drilling
▽ During Drilling

DESCRIPTION

FILL, recycled asphalt.

frozen to 0.6 m.

FILL (glacial till), clay silty, some sand, trace gravel, very stiff, low to medium plastic, moist, mottled grey and black, recycled asphalt pieces, organics.

SILT, firm to stiff, low to medium plastic, moist, brown.

GLACIAL TILL, clay, silty, some sand, trace gravel, very stiff, medium plastic, moist, brown, oxide stained, gypsum crystals.

hard below 4.8 m.

trace seepage at 6.0 m.

grey below 10.0 m.

SAMPLE TYPE	SPT (N) BLOWS/ 300 mm	WATER CONTENT (%)	LIQUID LIMIT (%)	PLASTIC LIMIT (%)	UNIT WEIGHT (kN/m³)	COMPRESSIVE STRENGTH (kPa)	POCKET PEN. (kg/cm²)
⧄		8.0					
⧄		10.1	29	11			
⧄		15.9					2.5
⧄		18.7					3.0
☒	6	19.3					
⧄		10.8					3.0
☒	26	10.2			22.4		4.5
⧄		10.5					4.5
☒	36	13.3			21.2		
⧄		12.4					4.5

DEPTH (m): 0 1 2 3 4 5 6 7 8 9 10 11 12

NOTES:
1. Borehole open and dry Immediately After Drilling.

SHEET 1 OF 1

05-13-2026 Y:\2026 Projects\23764 - Geo, Proposed Building, North Battleford Landfill, North Battleford\Drafting\Dwg\BH2-23764.bo

P.MACHIBRODA ENGINEERING LTD.

BOREHOLE 26-3

DRAWING NUMBER: 23764-4

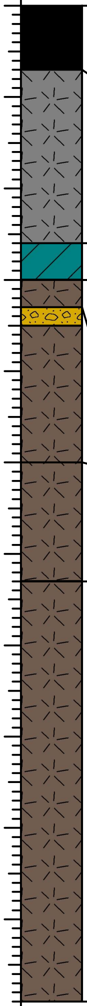
PROJECT: PROPOSED POLE BUILDING
NORTH BATTLEFORD LANDFILL

LOCATION: NORTH BATTLEFORD, SK

NORTHING (m): 5852329 **EASTING (m):** 685581 **ELEVATION (m):** 562.2 (+/-0.5 m) **DATE DRILLED:** APR 10/26

SAMPLE TYPE: ⧄ CUTTINGS ☒ SPLIT SPOON ■ SHELBY TUBE

WATER LEVELS
▼ After Drilling
▽ During Drilling



DESCRIPTION

FILL, recycled asphalt.
frozen to 0.6 m.

FILL (glacial till), clay silty, some sand, trace gravel, very stiff, medium plastic, moist, mottled grey and black, recycled asphalt pieces, organics.

CLAY, firm, medium plastic, moist, brown.

GLACIAL TILL, clay, silty, some sand, trace gravel, very stiff, low to medium plastic, moist, brown, oxide stained, gypsum crystals.
sandy gravel seam, seepage, sloughing 3.3 to 3.5 m.
hard below 4.5 m.

GLACIAL TILL, sand, silty, trace clay, trace gravel, dense, well graded, fine to coarse grained, moist, brown, oxide stained, seepage, sloughing.

GLACIAL TILL, clay, silty, some sand, trace gravel, hard, low to medium plastic, moist, brown, oxide stained, gypsum crystals.

SAMPLE TYPE	SPT (N) BLOWS/ 300 mm	WATER CONTENT (%)	LIQUID LIMIT (%)	PLASTIC LIMIT (%)	UNIT WEIGHT (kN/m³)	COMPRESSIVE STRENGTH (kPa)	POCKET PEN. (kg/cm²)
⧄		5.8					
⧄		9.9					
☒	24	12.4			19.5		
⧄		25.4					0.5
⧄		11.1					
☒	28	11.5			22.1		
⧄		11.5					
☒	45	11.2			22.0		
⧄		11.2					4.5
☒	42	12.8			22.0		

▽

NOTES:
1. Borehole open with water level at 9.2 m Immediately After Drilling.

SHEET 1 OF 1

05-13-2026 Y:\2026 Projects\23764 - Geo, Proposed Building, North Battleford Landfill, North Battleford\Drafting\Dwg\BH3-23764.bo

APPENDIX A

Explanation of Terms on Borehole Logs

CLASSIFICATION OF SOILS

Coarse-Grained Soils: Soils containing particles that are visible to the naked eye. They include gravels and sands and are generally referred to as cohesionless or non-cohesive soils. Coarse-grained soils are soils having more than 50 percent of the dry weight larger than particle size 0.080 mm.

Fine-Grained Soils: Soils containing particles that are not visible to the naked eye. They include silts and clays. Fine-grained soils are soils having more than 50 percent of the dry weight smaller than particle size 0.080 mm.

Organic Soils: Soils containing a high natural organic content.

Soil Classification By Particle Size

Soil Type	Particles of Size
Clay	< 0.002 mm
Silt	0.002 – 0.060 mm
Sand	0.06 – 2.0 mm
Gravel	2.0 – 60 mm
Cobbles	60 – 200 mm
Boulders	>200 mm

TERMS DESCRIBING CONSISTENCY OR CONDITION

Coarse-grained soils: Described in terms of compactness condition and are often interpreted from the results of a Standard Penetration Test (SPT). The standard penetration test is described as the number of blows, N, required to drive a 51 mm outside diameter (O.D.) split barrel sampler into the soil a distance of 0.3 m (from 0.15 m to 0.45 m) with a 63.5 kg weight having a free fall of 0.76 m.

Compactness Condition	SPT N-Index (blows per 0.3 m)
Very loose	0-4
Loose	4-10
Compact	10-30
Dense	30-50
Very dense	Over 50

Fine-Grained Soils: Classified in relation to undrained shear strength.

Consistency	Undrained Shear Strength (kPa)	N Value (Approximate)	Field Identification
Very Soft	<12	0-2	Easily penetrated several centimetres by the fist.
Soft	12-25	2-4	Easily penetrated several centimetres by the thumb.
Firm	25-50	4-8	Can be penetrated several centimetres by the thumb with moderate effort.
Stiff	50-100	8-15	Readily indented by the thumb, but penetrated only with great effort.
Very Stiff	100-200	15-30	Readily indented by the thumb nail.
Hard	>200	>30	Indented with difficulty by the thumbnail.

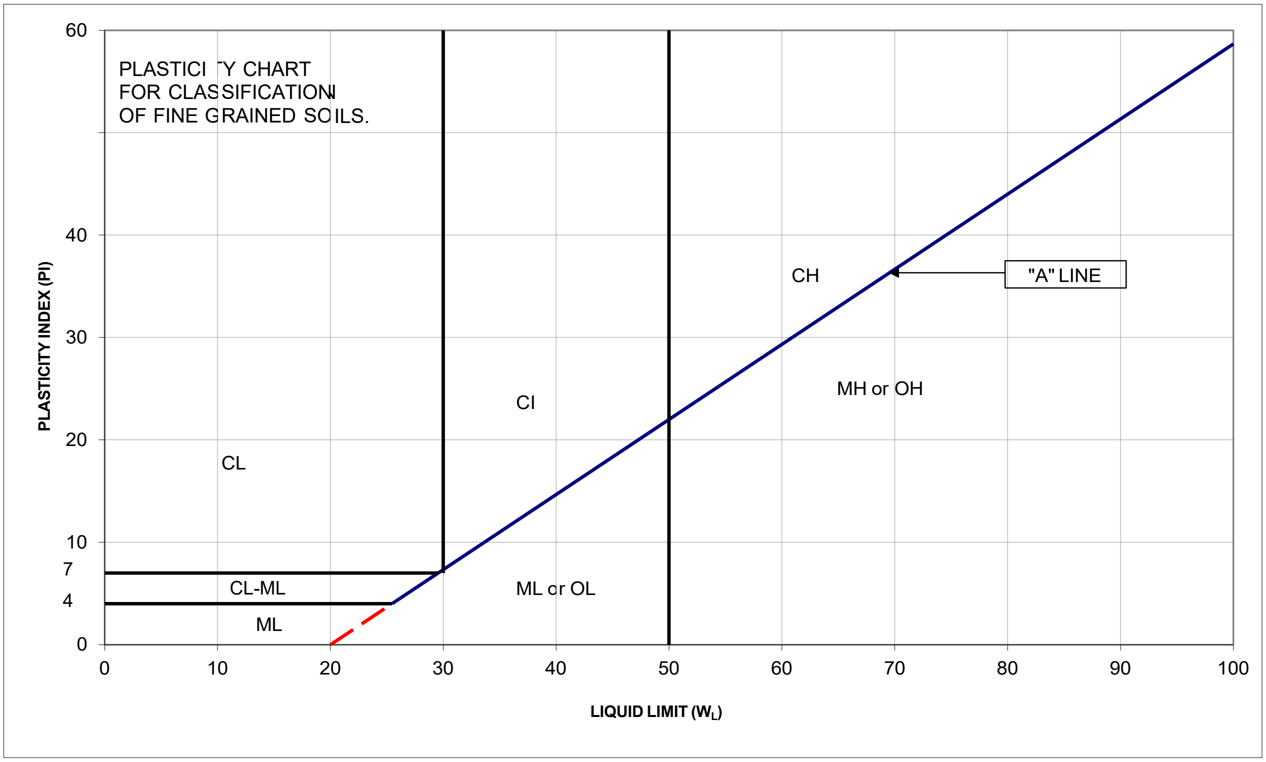
Organic Soils: Readily identified by colour, odour, spongy feel and frequently by fibrous texture.

DESCRIPTIVE TERMS COMMONLY USED TO CHARACTERIZE SOILS

Poorly Graded - predominance of particles of one grain size.
Well Graded - having no excess of particles in any size range with no intermediate sizes lacking.
Mottled - marked with different coloured spots.
Nuggety - structure consisting of small prismatic cubes.
Laminated - structure consisting of thin layers of varying colour and texture.
Slickensided - having inclined planes of weakness that are slick and glossy in appearance.
Fissured - containing shrinkage cracks.
Fractured - broken by randomly oriented interconnecting cracks in all 3 dimensions

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

MAJOR DIVISION			GROUP SYMBOL	TYPICAL DESCRIPTION	LABORATORY CLASSIFICATION CRITERIA
HIGHLY ORGANIC SOILS			Pt	PEAT AND OTHER HIGHLY ORGANIC SOILS	STRONG COLOUR OR ODOUR AND OFTEN FIBROUS TEXTURE
COARSE-GRAINED SOILS(MORE THAN HALF BY WEIGHT LARGER THAN NO. 200 SIEVE SIZE)	GRAVELS More than half coarse fraction larger than No. 4 sieve size	CLEAN GRAVELS	GW	WELL-GRADED GRAVELS, GRAVEL-SAND MIXTURES <5% FINES	$C_u = \frac{D_{60}}{D_{10}} > 4$ $C_c = \frac{(D_{30})^2}{D_{60} \times D_{10}} = 1$ to 3
			GP	POORLY-GRADED GRAVELS AND GRAVEL-SAND MIXTURES <5% FINES	NOT MEETING ALL ABOVE REQUIREMENTS FOR GW
		DIRTY GRAVELS	GM	SILTY GRAVELS, GRAVEL-SAND-SILT MIXTURES >12% FINES	ATTERBERG LIMITS BELOW "A" LINE OR PI < 4
			GC	CLAYEY GRAVELS, GRAVEL-SAND-CLAY MIXTURES >12% FINES	ATTERBERG LIMITS ABOVE "A" LINE WITH PI > 7
	SANDS More than half coarse fraction smaller than No. 4 sieve size	CLEAN SANDS	SW	WELL-GRADED SANDS, GRAVELLY SANDS MIXTURES <5% FINES	$C_u = \frac{D_{60}}{D_{10}} > 6$ $C_c = \frac{(D_{30})^2}{D_{60} \times D_{10}} = 1$ to 3
			SP	POORLY-GRADED SANDS OR GRAVELLY SANDS <5% FINES	NOT MEETING ALL GRADATION REQUIREMENTS FOR SW
		DIRTY SANDS	SM	SILTY SANDS, SAND-SILT MIXTURES >12% FINES	ATTERBERG LIMITS BELOW "A" LINE OR PI < 4
			SC	CLAYEY SANDS, SAND-CLAY MIXTURES >12% FINES	ATTERBERG LIMITS ABOVE "A" LINE WITH PI >7
FINE-GRAINED SOILS (MORE THAN HALF BY WEIGHT PASSING NO. 200 SIEVE SIZE)	SILTS Below "A" line on plasticity chart; negligible organic content		ML	INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTY SANDS OF SLIGHT PLASTICITY	$W_L < 50$
			MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS, FINE SANDY OR SILTY SOILS	$W_L > 50$
	CLAYS Above 'A" line on plasticity chart; negligible organic content		CL	INORGANIC CLAYS OF LOW PLASTICITY, GRAVELLY, SANDY, OR SILTY CLAYS, LEAN CLAYS	$W_L < 30$
			CI	INORGANIC CLAYS OF MEDIUM PLASTICITY, SILTY CLAYS	$W_L > 30 < 50$
			CH	INORGANIC CLAYS OF HIGH PLASTICITY, FAT CLAYS	$W_L > 50$
	ORGANIC SILTS & ORGANIC CLAYS Below "A" line on plasticity chart		OL	ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY	$W_L < 50$
			OH	ORGANIC CLAYS OF HIGH PLASTICITY	$W_L > 50$



APPENDIX B

Grain Size Distribution
Test Results

P.MACHIBRODA ENGINEERING LTD.

AASHTO T 88: PARTICLE SIZE ANALYSIS OF SOILS

Project: Proposed Pole Building

Location: North Battleford Landfill, North Battleford, SK

Project No.: 23764

Date Tested: May 11, 2026

Borehole No.: 26-2

Sample No.: 12

Depth (m): 1.0

Sieve Analysis:

Sieve	Diameter mm	% Finer
1.5"	38.100	100
1"	25.400	100
3/4"	19.100	100
1/2"	12.700	100
3/8"	9.500	100
# 4	4.750	97
# 10	2.000	94
# 20	0.850	85
# 40	0.425	71.8
#60	0.250	64.5
# 100	0.150	58.0
# 200	0.075	49.7

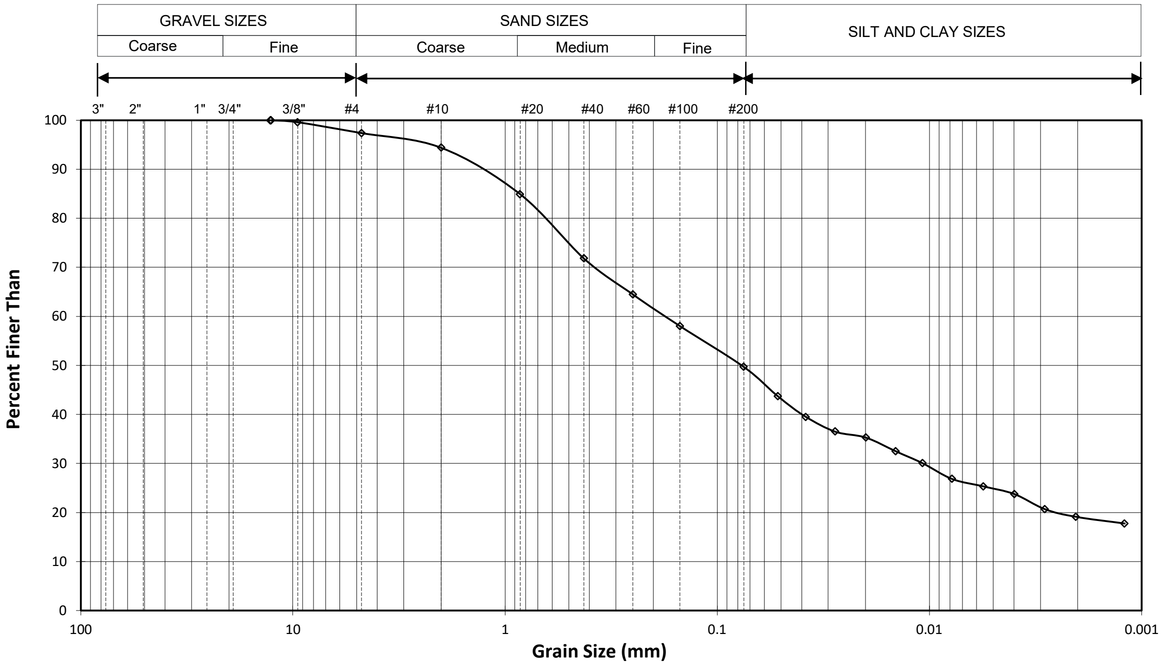
Hydrometer Analysis:

	Diameter mm	% Finer
Dispersing Agent:	0.0518	43.7
Sodium Hexametaphosphate	0.0382	39.5
	0.0278	36.5
	0.0199	35.3
	0.0144	32.5
	0.0108	30.1
	0.0078	26.8
	0.0056	25.3
	0.0040	23.8
	0.0029	20.7
	0.0020	19.1
	0.0012	17.7

Material Description:

% Gravel Sizes	% Sand Sizes	% Silt Sizes	% Clay Sizes
3	47	31	19

Remarks:



CERTIFIED BY CCIL
Canadian Council of Independent Laboratories
For specific tests as listed on www.ccil.com

Drawing No.

Appendix B-1

WE CERTIFY TESTING PROCEDURES ARE IN ACCORDANCE
WITH AASHTO T 88 STANDARD
P. MACHIBRODA ENGINEERING LTD.

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