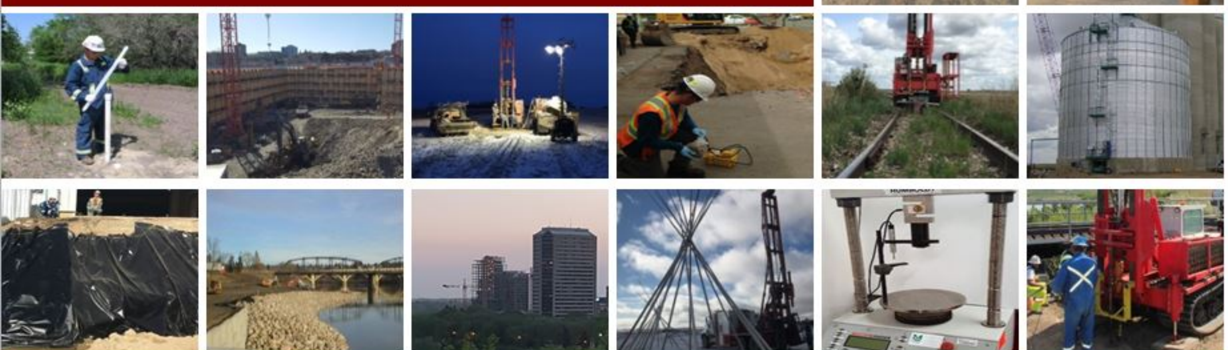
GEOTECHNICAL INVESTIGATION

PROPOSED COMMERCIAL DEVELOPMENT
LAVOIE STREET
BEAUVAL, SASKATCHEWAN
PMEL FILE NO. 21273
JUNE 13, 2024



PREPARED FOR:
Metis Nation – Saskatchewan Secretariat Inc.

ATTENTION: Melissa Pederson, A. Sc. T., Acting Director of Infrastructure

P.MACHIBRODA
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DOWN TO EARTH SOLUTIONS

P.MACHIBRODA ENGINEERING LTD.

PROJECT: Geotechnical Investigation
Proposed Commercial Development
Lavoie Street
Beauval, Saskatchewan
PMEL File No. 21273
June 13, 2024

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TABLE OF CONTENTS

LIST OF TABLES

LIST OF DRAWINGS

LIST OF FIGURES

LIST OF APPENDICES

1 INTRODUCTION

1.1 GENERAL

The following report has been prepared on the subsurface soil conditions existing at the site of the proposed commercial development to be constructed in Beauval, Saskatchewan.

The terms of reference for this investigation were presented in P. Machibroda Engineering Ltd. (PMEL) Proposal No. 21273, dated February 12, 2024. Authorization to proceed with this investigation was provided in the signed consulting agreement between Metis Nation – Saskatchewan Secretariat Inc. and PMEL, dated May 1, 2024.

1.2 SITE LOCATION AND DESCRIPTION

The proposed commercial development is located at the southwest corner of the intersection of Lavoie Street and Smith Avenue in Beauval, Saskatchewan. The site is undeveloped but was formerly occupied by an outdoor skating rink (has since been removed). The former rink area is a low-lying area that was flooded due to heavy rainfall at the time of the field investigation (refer to Figure 1). A Site Plan showing the location of the study area has been shown on Drawing No. 21273-1.

FIGURE 1 PHOTOGRAPH OF FLOODED FORMER RINK AREA

2 FIELD INVESTIGATION

The field investigation was conducted on May 22, 2024.

The coordinates at each borehole location were determined using a handheld GPS. The ground surface elevation at each borehole location was referenced to the top bolt of a fire hydrant, located approximately as shown on the Site Plan, Drawing No. 21273-1. A datum elevation of 100.0 m was assumed for the top of the bolt.

Nine boreholes, located as shown on the Site Plan, Drawing No. 21273-1, were dry drilled using our track-mounted, continuous flight auger drilling rig. The boreholes were 150 mm in diameter and extended to depths of 3.0 to 10.9 m below the existing ground surface. Borehole logs, as shown on the attached Drawing Nos. 21273-2 to 10, 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.

Monitoring wells (50 mm diameter, slotted PVC pipe) were installed in BH's 24-6 and 24-8 for groundwater monitoring purposes. Groundwater monitoring was completed on May 24, 2024.

3 SOIL AND GROUNDWATER CONDITIONS

3.1 SOIL PROFILE

The general soil profile consisted of sand and/or silt (to a depth of about 0.9 to 3 m) overlying sand and/or silt till (to a depth of about 7.8 to 8 m), followed by sand, which extended to a depth of at least 10.9 m, the maximum depth investigated.

The silt was firm to stiff, low to medium plastic and moist to wet. The surficial sand was loose and wet. It is noted that the silt and sand deposits were wet in some areas due to heavy rainfall just prior to the field investigation (i.e., flooded/ponded water). The till was compact to dense/hard, non-plastic/low plastic, damp to moist and contained cobbles/boulders throughout. The deeper sand deposits were very dense and damp.

3.2 GROUNDWATER CONDITIONS, SLOUGHING

Perched groundwater conditions were encountered in the surficial sand and silt deposits (i.e., ponded water/flooding due to heavy rainfall events just prior to the field investigation). The monitoring wells installed in BH's 24-6 and 24-8 were dry (i.e., groundwater greater than 3 m below grade) on May 24, 2024. Higher water levels should be expected during and/or following spring snowmelt and/or periods of precipitation.

3.3 Cobblestones and Boulders

Cobblestones and/or boulders were encountered throughout the till and sand deposits, as shown on the borehole logs. Auger refusal was encountered on cobbles/boulders at a depth of 7 m in BH’s 24-1 and 24-3.

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 anticipated.

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 volume of soil excavated/number and depth of piles installed.

4 Laboratory Analysis

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

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. 21273-2 to 10, inclusive.

The results of the grain size distribution analyses have been shown plotted 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 commercial development will be constructed at the site (details are unknown at this time). It is anticipated that the development will include at-grade buildings and associated traffic areas.

The subsurface soil conditions consisted of sand and/or silt overlying till, followed by sand. Cobbles/boulders were encountered throughout the sand and till deposits during test drilling. The groundwater table was situated below a depth of 3 m on May 24, 2024. Perched groundwater conditions were encountered in the surficial sand and silt deposits (i.e., ponded water/flooding due to heavy rainfall events just prior to the field investigation).

The subgrade soils are considered frost susceptible, and the potential depth of frost penetration could range from approximately 2.2 to 3.2 m, depending on surface cover, severity of winter and heat loss affects beneath/adjacent buildings.

A footing foundation based within naturally deposited, undisturbed till or sand should perform satisfactorily.

Alternately, a deep foundation system consisting of drilled, cast-in-place concrete piles could be considered. Cobbles/boulders will cause construction difficulties (potentially severe) and coring will likely be required to advance past boulders at some locations.

Recommendations have been prepared for site preparation; limit states resistance factors and serviceability; footings; deep foundations; grade-supported concrete slabs; foundation concrete; site classification for seismic site response; and, traffic structures.

5.2 SITE PREPARATION

All vegetation, topsoil, organics and deleterious materials should be removed from the construction area. Staining and root intrusion from the original/overlying organic material and roots may be encountered during excavation within the subsurface mineral soils. If these conditions are suspected, a representative of the Geotechnical Consultant should inspect the site during excavation to verify the depth of organic topsoil which should be removed in preparation of the site for construction.

The general intent of initial site preparation is to make the subgrade suitably stable for construction activities. It is recommended that the subgrade soils within the development footprint are compacted to the following densities:

Building Areas	96 percent standard Proctor density at optimum moisture content;
Traffic Areas	96 percent standard Proctor density at optimum moisture content;
Landscape Areas	90 percent standard Proctor density at optimum moisture content.

Soils which meet the required compaction level should be stable to support construction activities. It is anticipated that conventional site preparation (scarifying, moisture conditioning and re-compacting the soils) will suffice at this site. Soils which are unstable during site preparation and fail to achieve the required compaction will require additional treatment, which may include over-excavation and replacement and/or geosynthetic stabilization. The need for additional treatment should be reviewed by the Geotechnical Consultant during the field construction with respect to the actual conditions and project requirements.

In areas with variable subgrade soils, proof rolling may be an acceptable alternative to density testing and should be reviewed by the Geotechnical Consultant.

All proposed fill materials should be approved by the Geotechnical Consultant prior to placement. The fill should be placed in thin lifts (maximum 150 mm loose) and uniformly compacted to the above-mentioned densities.

Fill will be susceptible to settlement, the magnitude of which will be directly related to the level of compaction and fill thickness. Well compacted fills will settle a small percentage of the fill thickness whereas poorly compacted fills can settle appreciably, particularly if frozen soils are utilized/incorporated in the fill.

The site should be graded to provide positive site drainage away from all work areas and structures prior to, during and following construction.

This report has been prepared on the premise that significant alterations to the site will not occur (i.e., appreciable cut/fill activities). If appreciable quantities of fill will be placed on the site, settlement of the fill and underlying soils will occur which may affect the long-term performance of foundations, slabs, pavements etc.

If site alterations are planned as part of site development, PMEL should be contacted to assess the impact this may have on the design recommendations and proposed site development. Based on the magnitude of site alterations, the design recommendations may need to be amended.

5.3 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.

Shallow foundations:

- Compressive Resistance, $\Phi = 0.5$
- Sliding, Based on Friction (c=0), $\Phi = 0.8$

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 Sections 5.4 and 5.5.3.

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 5.5.4.

5.4 FOOTINGS

A footing foundation based within naturally deposited, undisturbed till or sand should perform satisfactorily. If the foundation is constructed during freezing conditions, the subgrade soil at the design footing elevation must be protected from freezing. If it is not practical to keep the subgrade from freezing then a deep foundation system should be constructed.

The following minimum recommendations should be incorporated into the design of a footing foundation supporting a continually heated building. The recommendations are applicable to footings supporting vertical concentric loading only; footings subject to eccentric/unbalanced loading will require additional assessment.

1. Footings should be founded on naturally deposited, undisturbed till or sand. Footings should not be based on silt soils.

2. For a permanently heated, at-grade structure where heat loss to the foundation is permitted, the footings should be based at a minimum depth of 2 m below finished grade. This minimum depth is applicable only where the building envelope insulation is designed to allow heat loss to the foundation. If insulation is placed beneath the floor slab, an uninsulated strip width of at least 1 m is recommended adjacent to all exterior grade beams/foundation caps. In unheated areas and/or where heat loss from the building to the foundation is not allowed, footings should be based below the potential depth of frost penetration (i.e., 3.2 m) or protected against frost action with strategically placed extruded polystyrene insulation (PMEL can provide insulation recommendations upon request if shallower foundation depths than recommended are desirable).

3. The footing excavations should be conducted strategically to minimize the lateral extent of the excavation under the floor slab as much as possible. If personnel are entering the excavations, sideslopes should be no steeper than 1H:1V, whereas if personnel are not entering the excavations, steeper sideslopes may be feasible.

4. Footings based on naturally deposited, undisturbed soil may be designed to exert an unfactored ULS bearing pressure of 600 kPa and an SLS bearing pressure of 150 kPa (to limit settlements to less than 25 mm). A maximum spread footing dimension of 3 m and a maximum strip footing width of 1 m was considered to determine the SLS bearing pressure; for larger footing sizes, an updated settlement analysis will be required. Both ULS and SLS should be assessed and the condition which results in the larger foundation should be utilized for design (i.e., limiting condition).

5. A representative of the Geotechnical Consultant should inspect the footing excavations prior to construction of the footings to verify that adequate soil conditions exist. Placement of a mud slab over the prepared foundation level would be beneficial to provide protection from disturbance.

6. A minimum strip footing width of 500 mm is recommended. A minimum dimension of 1,000 mm is recommended for square and rectangular footings.

7. If the subgrade soil is disturbed during excavation below the design depth, then the disturbed soil should be removed to an undisturbed, level surface. Fill, required to raise the subgrade elevation to the underside of the footings, should be concrete or fillcrete.

8. Footings should not be constructed on desiccated, frozen or wet subgrade soil. Frost should not be allowed to penetrate beneath the footings prior to, during or after construction.

9. The finished grade should be landscaped to provide for positive site drainage away from the structure.

5.5 DEEP FOUNDATIONS

5.5.1 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 2.2 m (lower bound) to 3.2 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 suspectable to frost jacking and must be designed to resist frost jacking forces resulting from the upper bound frost penetration depth.

Interior piles which are never exposed to frost action (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., 2.2 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., 3.2 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.

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 (point 2 above), the structural designer should consider the unfactored ULS resistance values presented in the following 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; 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.

5.5.2 Drilled, Cast-in-Place Concrete Piles

Cobbles/boulders will cause construction difficulties (potentially severe) and coring will likely be required to advance past boulders at some locations. High powered drilling equipment is recommended due to the presence of hard / dense soil conditions and cobbles/boulders.

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 below.

Table I Shaft Resistance (Drilled Piles)

Zone / Depth (m) [1]	Shaft Resistance (kPa)	
	Unfactored ULS	SLS
Neglect Zone [2]	0	0
Below Neglect Zone to 4.5	75	30
Below 4.5	125	50

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. The shaft resistance in the zone from finished ground surface to a depth of 2 m below finished ground surface (i.e., neglect zone) should be ignored in terms of axial capacity.

3. Minimum pile lengths should take into account the depth required to resist frost action. Piles exposed to frost action may need to be longer and should be designed to resist frost jacking forces (refer to Section 5.5.1).

4. Piles should be reinforced to withstand all axial and lateral forces within the pile.

5. 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.

6. The pile holes should be filled with concrete as soon as practical after drilling.

7. 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.

8. Due to the hard/dense nature of the subsurface deposits and the presence of cobbles/boulders, high-powered piling equipment is recommended.

9. A minimum centre-to-centre pile spacing of not less than three pile diameters is recommended.

10. A representative of the Geotechnical Consultant should inspect and document the installation of the drilled, cast-in-place concrete piles.

5.5.3 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 straight shaft 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.

5.5.4 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 II.

TABLE II ESTIMATED COEFFICIENTS OF HORIZONTAL SUBGRADE REACTION

Depth (m)	Coefficient of Horizontal Subgrade Reaction, Ks, (kN/m^3)
0 to 1.5D	0
1.5D to 2	5,000/D
2 to 4.5	25,000/D
Below 4.5	40,000/D

Where D = pile diameter (m). For large diameter piles (i.e., exceeding 1 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.

5.5.5 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 150 mm of net void space between the underside of the grade beam 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.

5.6 Grade-Supported Concrete Slabs

5.6.1 Slabs in Heated Areas

The near-surface subgrade soils consisted of sand and/or silt. Grade-supported concrete slabs should perform satisfactorily at this site.

The following minimum provisions should be incorporated into the design of conventional, heated, grade-supported, cast-in-place, at-grade reinforced concrete slabs subject to light loading.

1. Prepare the site in accordance with Section 5.2. Level and compact the upper 150 mm of subgrade soil to 96 percent of standard Proctor density at optimum moisture content.

2. Soft subgrade areas should be excavated and replaced with suitable soil compacted to 96 percent of standard proctor density at optimum moisture content. High-strength geogrid/geotextile may be required to provide soil stabilization and separation where soft/wet soil conditions are encountered. The need for special measures (i.e., over-excavation, geotextile, geogrid, and/or additional gravel fill) in soft/wet areas must be subject to review by the Geotechnical Consultant during field construction.

3. If fill is required to raise the existing subgrade surface to the design slab elevation, locally available soils are considered suitable; sand or till are preferable to silt, and the soils may require moisture-conditioning. The fill should be placed in thin lifts (150 mm loose, maximum) and uniformly compacted to 98 / 96 percent of standard Proctor density at optimum moisture content for sand or till / silt soils, respectively.

4. The uppermost portion of the fill immediately below the slab should consist of crushed granular base course material (150 mm minimum).

5. Isolate the slab from foundation walls, columns, etc., by means of separation joints.

6. Reinforce the concrete slab and articulate the slab at regular intervals to provide for controlled cracking.

7. Provide positive site drainage away from the proposed structure. Extend downspouts at least 3 m away from the foundation.

8. Floor slabs should not be constructed on desiccated, wet, or frozen subgrade soil or base.

9. Frost should not be allowed to penetrate beneath the floor slab just prior to, during or after construction.

10. A soil gas membrane (i.e., radon gas and moisture resistant) should be installed between the underside of the floor slab and the granular fill.

5.6.2 Slabs Exposed to Freezing Conditions

Grade-supported concrete slabs exposed to freezing conditions (i.e., exterior slabs/sidewalks, etc.) will be subject to differential movements associated with frost action. The potential for differential movements associated with frost action can be minimized by placing sub-horizontal extruded polystyrene insulation below the slabs/sidewalks.

Where applicable, the insulation should butt-up to the foundation to direct heat to the underside of the slab. The insulation should have a minimum thickness of 125 mm and should extend sub-horizontally to a minimum distance of 2.1 m beyond the outer edges of the slab. If differential movements cannot be tolerated, the slab should be structurally supported on piles.

5.6.3 Soil Gas (Radon) Mitigation

Within human-occupied areas, measures to mitigate radon gas should be incorporated into the overall design of the proposed building. This could include a sub-slab depressurization system (i.e., radon rock, drainage pipes, suction pit etc.), radon gas membrane and/or adequate ventilation system. The design of the radon mitigation system should be undertaken by qualified designers and should be in accordance with all applicable regulations and building codes. If a sub-slab depressurization system is installed, the design should be reviewed by the Geotechnical Consultant.

If radon rock is incorporated into the design, it should be placed below the granular base course layer of the floor slab structure. The radon rock should be fully encapsulated in non-woven geotextile, capable of transmitting a flow of not less than 50 litres per second per square metre (ASTM D-4491).

5.7 Foundation Concrete

The results water-soluble sulphate testing on representative soil samples recovered from the subject site have been summarized in Table III.

TABLE III WATER-SOLUBLE SULPHATE TEST RESULTS

Borehole No.	Depth (m)	Soil Type	Water Soluble Sulphate (%)	Class of Exposure	Degree of Sulphate Exposure
24-5	3	Till	<0.05	--	Negligible
24-7	6	Till	<0.05	--	Negligible

As shown in Table III, the measured sulphate concentration of the tested soils was less than 0.05 percent, which is considered negligible in terms of potential degree of sulphate attack.

Based on the test results, general use cement (CSA designation GU) may be used 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

5.8 SITE CLASSIFICATION FOR SEISMIC SITE RESPONSE

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

5.9 TRAFFIC STRUCTURES

5.9.1 ASPHALT CONCRETE PAVEMENT STRUCTURE THICKNESS

The near surface subgrade soils at the site consisted of sand and/or silt. Based on the results of the laboratory analysis, the CBR (California Bearing Ratio) rating of the compacted subgrade soil was estimated to be in the order of 4.

Design traffic volumes were not provided to PMEL for the subject site. As such, the assumed traffic volumes presented in Table IV were utilized to develop the pavement structures presented in the following section.

TABLE IV ASSUMED TRAFFIC VOLUME

Truck Traffic Volume	Equivalent Single Axle Load (ESAL)	Approximate Equivalent Annual Average Daily Truck Traffic (AADTT)[1] – Single Unit Trucks
Low	40,000	Minimal to None
Moderate	120,000	5
High	700,000	35

[1] AADTT based on single unit trucks with a load equivalency factor of 1.2 (as per AASHTO 1993) and pavement design life of 15 years. The AADTT provided is for conceptual purposes only and will vary depending on actual truck traffic types (i.e., single unit trucks, tractor semi-trailer combinations, super B trucks, etc.).

The following pavement structures are recommended for pavement structures subjected to low, moderate, or high truck traffic volumes, consistent with the assumed traffic volumes presented in Table IV.

TABLE V THICKNESS DESIGN FOR PAVEMENT STRUCTURES

Pavement Structure Layer	Thickness (mm) Truck Traffic Volume		
	High	Moderate	Low
Asphalt Concrete	120	85	65
Granular Base	175	150	125
Granular Sub-Base	215	175	150
Geotextile/Geogrid [1]	As Required	As Required	As Required
Prepared Subgrade	(150)	(150)	(150)
Total Thickness (mm)	**510**	**410**	**340**

[1] Geogrid/geotextile may be required where soft/wet/loose subgrade soil conditions are encountered.

Notes:

1. If the parking lot (or portions of the parking lot) will be subject to traffic volumes and/or truck types varying, or in excess of, those presented in Table IV, PMEL should be notified to review our recommendations. A detailed traffic volume analysis may be required. Depending on the actual traffic volume, the recommended pavement structure may be adjusted.

2. It should be noted that the low truck traffic pavement structure is the minimum pavement structure recommended for the parking/driving areas.

3. Traffic should be appropriate for the pavement structure (i.e., do not allow heavy traffic on light structures) or premature distress/failure may occur.

5.9.2 PAVEMENT STRUCTURE CONSTRUCTION

The following minimum recommendations should be incorporated into the construction of the pavement structures.

1. Prepare the site in accordance with Section 5.2, Site Preparation. Level and compact the upper 150 mm of subgrade soil to 96 percent of standard Proctor density at optimum moisture content.

2. If minor site grading is required to create a level subgrade surface, locally available soils may be utilized as fill. The fill should be placed in thin lifts (150 mm loose, maximum) and uniformly compacted to 96 percent of standard Proctor density at optimum moisture content. If significant site grading is required, PMEL should be contacted to reassess the design recommendations.

3. It is recommended that PMEL conduct a visual site assessment and proof roll on the prepared subgrade prior to construction of the traffic structures presented in Table V. Remediation (i.e., over-excavation and replacement or geotextile/geogrid) will be required for areas of the roadway where deflection/rutting of the subgrade is observed at the time of the proof roll. The amount of over-excavation required will be dependent upon the severity of the deficiency observed. Recommendations for remediation, if required, would be provided based on the field conditions observed at the time of the visual assessment.

4. All granular fill placed above the prepared subgrade should be placed in thin lifts (150 mm loose) and compacted to 98 percent of standard Proctor density at optimum moisture content. The granular sub-base and base course material should meet the aggregate gradation requirements provided in Table VI. If available/economical, Type 31 base and Type 6 sub-base materials are considered preferable, particularly for gravel-surfaced structures.

TABLE VI AGGREGATE GRADATION REQUIREMENTS

Grain Size (mm)	Percent Passing			
	Type 33 Base-Course [1]	Type 31 Base-Course [1]	Type 8 Sub-Base Course [2]	Type 6 Sub-Base Course [2]
50.0	--	--	100	100
31.5	--	100	--	--
18.0	100	75 – 90	--	--
12.5	75 – 100	65 – 83	--	--
5.0	50 – 75	40 – 69	--	--
2.0	32 – 52	26 – 47	0 – 90	0 – 80
0.900	20 – 35	17 – 32	--	--
0.400	15 – 25	12 – 22	0 – 60	0 – 45
0.160	8 – 15	7 – 14	0 – 25	0 – 20
0.071	6 – 11	6 – 11	0 – 15	0 – 6
Plasticity Index (%)	0 – 6	0 – 7	0 – 6	0 – 6
CBR (Min)	65	65	20	20
% Fracture (Min)	50	50	--	--

[1] MHI base course.
[2] MHI sub-base course.

5. Positive surface drainage is recommended to reduce the potential for moisture infiltration through the pavement structure.

6. Surface water should be prevented from seeping back under the outer edges of the traffic structure.

7. Where possible, grades should be designed such that the granular materials can freely discharge into ditches or into a sub-surface drainage system; this will provide a capillary break to maintain an unsaturated condition in the overlying traffic structure (this is especially important in low-lying/wetland areas).

8. Periodic maintenance such as crack sealing will be required for asphalt concrete pavement.

9. If concrete catch basins are installed, a series of small holes (25 mm diameter minimum) should be drilled through the catch basin to allow for drainage of free water which may collect adjacent to the catch basin. A layer of non-woven geotextile should be used to encapsulate the catch basin and the surrounding (free-draining) backfill material to prevent clogging of the drainage holes.

10. Damage to the pavement related to frost heave around structures constructed within the pavement (i.e., manholes, curbs, backs of curbs, etc.) may occur. Consideration could be given to constructing 3H:1V frost tapers (constructed with granular fill) at these structures which should reduce the potential for pavement cracking around the structure. Frost tapers are also recommended at transitions between high, moderate, and low truck traffic pavement structures.

11. Damage to the pavement related to a change in stiffness where asphalt concrete pavements transition into concrete (i.e., at the location of the loading dock/pump island/garbage bin slab/etc.) is common. To reduce the potential for damage, a stiffness transition zone could be constructed at the transition between pavement types. The suggested configuration of the stiffness transition zone has been shown in Figure 2.

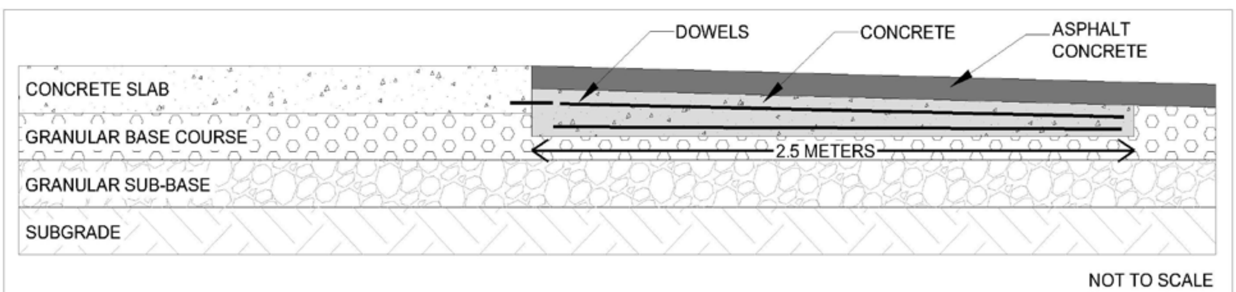

FIGURE 2 STIFFNESS TRANSITION

If soil embankments are constructed, the following additional recommendations should be considered.

1. All common borrow used for embankment construction should consist of imported granular material or locally available soils; sand or till are preferable to silt.

2. Positive surface drainage is recommended to minimize the potential for moisture infiltration into the subgrade soil. Ditches and culverts should be provided where necessary to provide adequate site drainage. Surface water should be prevented from seeping back under the outer edges of the road structure. The embankments should be constructed with a shoulder height of at least 1.2 m above ditch bottom elevation.

3. For locally sourced borrow materials, embankment and ditch slopes should be no steeper than 3 Horizontal to 1 Vertical (3H : 1V).

4. Erosion protection is recommended for all embankment sideslopes. The slopes should be covered with topsoil and seeded to encourage vegetation growth. Alternately, erosion control products or hydromulch could be installed.

5. The final road grade should be elevated a minimum of 600 mm above the average terrain to minimize snow accumulation on the road.

5.9.3 Gravelled Traffic Structure Considerations

If gravel surfacing is utilized instead of asphalt concrete, the structure thicknesses presented in Table V should suffice if site drainage is adequate and regular maintenance/grading is completed to maintain the traffic structure at the desired condition. Type 31 base course is recommended for the driving surface. If drainage is insufficient and maintenance is inadequate, premature traffic structure failure may occur. If a lesser structure is utilized, more frequent maintenance will be required, and premature failure may occur.

If additional longevity/lesser maintenance is desired, the structure thickness should be increased and geosynthetics should be utilized at the base of the structure to provide material separation. PMEL can review the proposed gravelled traffic structures upon request to confirm suitability for the intended usage.

In staging/laydown areas subject to very infrequent traffic, a thin wearing course of traffic gravel may suffice provided that site drainage is sufficient, adequate maintenance is performed and heavy traffic does not travel on the gravel during soft subgrade conditions (i.e., during/following periods of precipitation and while soils are softened during spring thaw).

6 Limitations

The presentation of the summary of the borehole logs and design recommendations has been completed as authorized. Nine, 150 mm diameter boreholes were dry 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 location 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.

It is recommended that all monitoring wells should be decommissioned once they are no longer needed. PMEL will not accept any future liability associated with inadequate decommissioning of monitoring wells. Costs for decommissioning monitoring wells can be provided by PMEL upon request.

This report has been prepared for the exclusive use of Metis Nation – Saskatchewan Secretariat Inc. and their agents for specific application to the proposed commercial development be constructed in Beauval, 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.

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.

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.

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

P. MACHIBRODA ENGINEERING LTD.

PROFESSIONAL ENGINEER
C.C. ZUBROWSKI
MEMBER 12138
2024-06-13
YR. MN. DAY
SASKATCHEWAN

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	12138	

2024-06-13

Cory Zubrowski, P.Eng.

K. Pardoski

Kelly Pardoski, P.Eng.
CZ/JKP

DRAWINGS

KEY PLAN
NOT TO SCALE

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 ©2024, IMAGE ©2024 AIRBUS, (IMAGERY DATE: 10/14/23).
3. THIS DRAWING WAS COMPILED USING HANDHELD GPS EQUIPMENT (TRIMBLE UNIT 3, MODEL Geo 7X).
4. BENCHMARK: TOP BOLT OF FIRE HYDRANT AT EAST CORNER OF LOT. ASSUMED DATUM ELEVATION = 100.000 m.

LEGEND
- PMEL BOREHOLE
- PMEL BOREHOLE (MONITORING WELL IINSTALLED)
- BENCHMARK

CONSULTING
GEOENVIRONMENTAL
GEOTECHNICAL
ENGINEERS

P. MACHIBRODA ENGINEERING LTD.

806 – 48th STREET EAST
SASKATOON, SK
S7K 3Y4

DRAWING TITLE: SITE PLAN - BOREHOLE LOCATIONS		
PROJECT: PROPOSED COMMERCIAL DEVELOPMENT LAVOIE STREET, BEAUVAL, SK		
APPROVED BY: CZ	DRAWN BY: BS	DRAWING NUMBER: 21273-1
DATE: MAY, 2024	SCALE: AS SHOWN	

P.MACHIBRODA ENGINEERING LTD.

BOREHOLE 24-1

DRAWING NUMBER: 21273-2

PROJECT: PROPOSED COMMERCIAL DEVELOPMENT

LOCATION: LAVOIE STREET, BEAUVAL, SK

NORTHING (m): 6114258 **EASTING (m):** 333470 **ELEVATION (m):** 99.4 **DATE DRILLED:** MAY 22/24

SAMPLE TYPE: CUTTINGS SPLIT SPOON SHELBY TUBE

WATER LEVELS
- After Drilling
- During Drilling

DEPTH (m)	DESCRIPTION	SAMPLE TYPE	SPT (N) BLOWS/ 300 mm	WATER CONTENT (%)	LIQUID LIMIT (%)	PLASTIC LIMIT (%)	UNIT WEIGHT (kN/m³)	COMPRESSIVE STRENGTH (kPa)	POCKET PEN. (kg/cm²)
0 – 1.6	SILT, some clay, firm, low plastic, wet, brown.	Cuttings		21.9					
		Cuttings		30.4					
		Cuttings		31.8					
1.6 – 4.7	GLACIAL TILL, sand and silt, trace clay, trace gravel, compact to dense, well graded, fine to coarse grained, moist, brown, cobbles and boulders.								
		Split Spoon	40 (rocks)	6.5					
		Cuttings		8.4					
4.7 – 7.0	GLACIAL TILL, silt, sandy, some clay, trace gravel, hard, low plastic, moist, brown, oxide stained. cobbles and boulders 5.0 to 6.0 m.								
		Split Spoon	50+ (rocks)	6.8					
	cobbles and boulders 6.5 to 7.0 m.								
7.0	Auger refusal on boulders at 7.0 m.								

Water level during drilling: approx. 2.2 m.

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

P.MACHIBRODA ENGINEERING LTD.

BOREHOLE 24-2

DRAWING NUMBER: 21273-3

PROJECT: PROPOSED COMMERCIAL DEVELOPMENT

LOCATION: LAVOIE STREET, BEAUVAL, SK

NORTHING (m): 6114255 **EASTING (m):** 333482 **ELEVATION (m):** 98.9 **DATE DRILLED:** MAY 22/24

SAMPLE TYPE: CUTTINGS SPLIT SPOON SHELBY TUBE

WATER LEVELS
▼ After Drilling
▽ During Drilling

DEPTH (m)	STRATIGRAPHY	DESCRIPTION	SAMPLE TYPE	SPT (N) BLOWS/ 300 mm	WATER CONTENT (%)	LIQUID LIMIT (%)	PLASTIC LIMIT (%)	UNIT WEIGHT (kN/m³)	% FINES	POCKET PEN. (kg/cm²)	DEPTH (m)
0		SAND, silty, loose, well graded, fine to medium grained, wet, mottled brown/grey, rootlets, seepage, sloughing.			17.4	15	13		30		0
		SILT, some sand, trace clay, firm, low plastic, wet, mottled brown and grey.			19.9						
1					26.3						1
2		GLACIAL TILL, sand, silty, trace clay, trace gravel, compact, well graded, fine to coarse grained, moist, brown, oxide stained, cobbles and boulders			7.2						2
3					7.3						3
4											4
5											5
6											6
7											7
8											8
9											9
10											10
11											11
12											12

NOTES:
1. Borehole open with trace water Immediately After Drilling.

P.MACHIBRODA ENGINEERING LTD.

BOREHOLE 24-3

DRAWING NUMBER: 21273-4

PROJECT: PROPOSED COMMERCIAL DEVELOPMENT

LOCATION: LAVOIE STREET, BEAUVAL, SK

NORTHING (m): 6114243 **EASTING (m):** 333504 **ELEVATION (m):** 98.9 **DATE DRILLED:** MAY 22/24

SAMPLE TYPE: CUTTINGS SPLIT SPOON SHELBY TUBE

WATER LEVELS
- After Drilling
- During Drilling

DESCRIPTION

Description	Sample Type	SPT (N) Blows/300 mm	Water Content (%)	Liquid Limit (%)	Plastic Limit (%)	Unit Weight (kN/m³)	Compressive Strength (kPa)	Pocket Pen. (kg/cm²)
SAND, some silt, loose, well graded, fine to medium grained, wet, mottled grey/brown, seepage, sloughing.	Cuttings		17.0					
SILT, some clay, trace sand, firm to stiff, low plastic, moist to wet, mottled brown and grey, oxide stained.	Cuttings		19.2					
	Split Spoon	7	27.2					
GLACIAL TILL, sand and silt, trace gravel, trace clay, compact, well graded, fine to coarse grained, moist, brown, oxide stained.	Cuttings		10.0					
GLACIAL TILL, silty, sandy, some clay, trace gravel, hard, low plastic, moist, brown, oxide stained.	Split Spoon	50/150	8.3			22.4		
	Cuttings		11.6					
Auger refusal on boulders at 7.0 m.	Cuttings		10.9					

NOTES:
1. Borehole sloughed to 5.8 m Immediately After Drilling.

SHEET 1 OF 1

P.MACHIBRODA ENGINEERING LTD.

BOREHOLE 24-4

DRAWING NUMBER: 21273-5

PROJECT: PROPOSED COMMERCIAL DEVELOPMENT

LOCATION: LAVOIE STREET, BEAUVAL, SK

NORTHING (m): 6114242 **EASTING (m):** 333461 **ELEVATION (m):** 99.5 **DATE DRILLED:** MAY 22/24

SAMPLE TYPE: CUTTINGS SPLIT SPOON SHELBY TUBE

WATER LEVELS
After Drilling
During Drilling

DEPTH (m)	STRATIGRAPHY	DESCRIPTION	SAMPLE TYPE	SPT (N) BLOWS/ 300 mm	WATER CONTENT (%)	LIQUID LIMIT (%)	PLASTIC LIMIT (%)	UNIT WEIGHT (kN/m³)	COMPRESSIVE STRENGTH (kPa)	POCKET PEN. (kg/cm²)	DEPTH (m)
0		SILT, some clay, firm, medium plastic, moist to wet, mottled brown/grey, oxide stained. rootlets to 0.3 m.			26.4 23.1 28.5						0
1											1
2		GLACIAL TILL, sand and silt, trace gravel, trace clay, compact, well graded, fine to coarse grained, damp to moist, brown.			3.5 4.9						2
3											3
4											4
5											5
6											6
7											7
8											8
9											9
10											10
11											11
12											12

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

P.MACHIBRODA ENGINEERING LTD.

BOREHOLE 24-5

DRAWING NUMBER: 21273-6

PROJECT: PROPOSED COMMERCIAL DEVELOPMENT

LOCATION: LAVOIE STREET, BEAUVAL, SK

NORTHING (m): 6114239 **EASTING (m):** 333473 **ELEVATION (m):** 99.3 **DATE DRILLED:** MAY 22/24

SAMPLE TYPE: CUTTINGS SPLIT SPOON SHELBY TUBE

WATER LEVELS

After Drilling

During Drilling

DEPTH (m)	DESCRIPTION
0 – 0.3	SAND, silty, loose, well graded, fine to medium grained, wet, mottled black/grey, organics, rootlets.
0.3 – 2.0	SILT, some clay, stiff, medium plastic, moist, brown, oxide stained.
2.0 – 8.0	GLACIAL TILL, silt, sandy, trace gravel, trace clay, hard, low plastic, moist, brown, oxide stained. cobbles and boulders 2 to 3 m. damp below 4.5 m.

DEPTH (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)
0.2	Cuttings		23.1					
0.7	Cuttings		24.2					
1.8	Split Spoon	15	27.0					
3.0	Cuttings		15.4	17	12			
4.8	Split Spoon		9.4					
6.0	Cuttings		6.5					
7.8	Split Spoon	50/150	6.0					
8.0	Cuttings		8.6					

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

SHEET 1 OF 1

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P. MACHIBRODA ENGINEERING LTD.

BOREHOLE 24-6

DRAWING NUMBER: 21273-7

PROJECT: PROPOSED COMMERCIAL DEVELOPMENT

LOCATION: LAVOIE STREET, BEAUVAL, SK

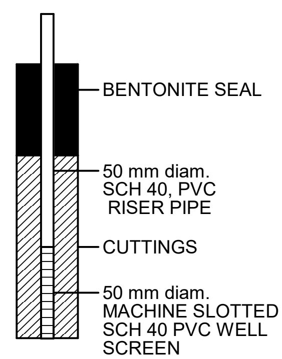
NORTHING (m): 6114227 **EASTING (m):** 333498 **ELEVATION (m):** 99.1 **DATE DRILLED:** MAY 22/24

SAMPLE TYPE: CUTTINGS SPLIT SPOON SHELBY TUBE

WATER LEVELS
- After Drilling
- During Drilling

DEPTH (m)	DESCRIPTION	SAMPLE TYPE	SPT (N) BLOWS/ 300 mm	WATER CONTENT (%)	LIQUID LIMIT (%)	PLASTIC LIMIT (%)	UNIT WEIGHT (kN/m³)	% FINES	POCKET PEN. (kg/cm²)
0 – 0.9	SILT, some clay, firm, low plastic, moist to wet, brown, oxide stained. stiff, moist below 0.5 m.			24.9	28	17		91	
				11.5					
0.9 – 3.0	GLACIAL TILL, sand and silt, trace clay, trace gravel, compact, well graded, fine to coarse grained, moist, brown, oxide stained, cobbles and boulders.			7.1					
				5.1					
				6.5					

MONITORING WELL: BH24-6
ELEV.: 100.1 m



NOTES:
1. Borehole open and dry Immediately After Drilling.
2. Recorded Monitoring Well groundwater Level Dry on May 24, 2024.

P.MACHIBRODA ENGINEERING LTD.

BOREHOLE 24-7

DRAWING NUMBER: 21273-8

PROJECT: PROPOSED COMMERCIAL DEVELOPMENT

LOCATION: LAVOIE STREET, BEAUVAL, SK

NORTHING (m): 6114227 **EASTING (m):** 333457 **ELEVATION (m):** 99.5 **DATE DRILLED:** MAY 22/24

SAMPLE TYPE: CUTTINGS SPLIT SPOON SHELBY TUBE

WATER LEVELS
- After Drilling
- During Drilling

DEPTH (m)	DESCRIPTION	SAMPLE TYPE	SPT (N) BLOWS/ 300 mm	WATER CONTENT (%)	LIQUID LIMIT (%)	PLASTIC LIMIT (%)	UNIT WEIGHT (kN/m³)	COMPRESSIVE STRENGTH (kPa)	POCKET PEN. (kg/cm²)
0–2	SILT, some clay, stiff, medium plastic, moist, brown, oxide stained. rootlets to 0.05 m	Cuttings		20.8					
		Cuttings		28.7					
		Split spoon	13	28.4			18.8		
2–5.5	GLACIAL TILL, sand and silt, trace clay, trace gravel, very dense, well graded, fine to coarse grained, damp to moist, brown, cobbles and boulders.	Cuttings		5.1					
		Split spoon	50/125	7.3					
5.5–8	GLACIAL TILL, silty, sandy, some clay, trace gravel, hard, low plastic, damp, brown, oxide stained.	Cuttings		6.6					
		Split spoon	50/100	7.7			22.8		
8–10.9	SAND, silty, very dense, well graded, fine to medium grained, damp, light brown, oxide stained. cobbles and boulders 8.0 to 9.0 m.	Cuttings		4.7					
		Split spoon	50/75	5.2					

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

SHEET 1 OF 1

06-12-2024 Y:\2024 Projects\21273 - Beauval 3 lot commercial development\Drafting\BH7-21273.bor

P.MACHIBRODA ENGINEERING LTD.

BOREHOLE 24-8

DRAWING NUMBER: 21273-9

PROJECT: PROPOSED COMMERCIAL DEVELOPMENT

LOCATION: LAVOIE STREET, BEAUVAL, SK

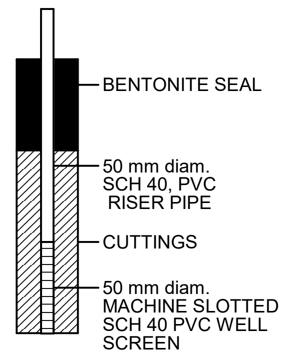
NORTHING (m): 6114217 **EASTING (m):** 333476 **ELEVATION (m):** 99.7 **DATE DRILLED:** MAY 22/24

SAMPLE TYPE: CUTTINGS SPLIT SPOON SHELBY TUBE

WATER LEVELS
▼ After Drilling
▽ During Drilling

DEPTH (m)	STRATIGRAPHY	DESCRIPTION	SAMPLE TYPE	SPT (N) BLOWS/ 300 mm	WATER CONTENT (%)	LIQUID LIMIT (%)	PLASTIC LIMIT (%)	UNIT WEIGHT (kN/m³)	COMPRESSIVE STRENGTH (kPa)	POCKET PEN. (kg/cm²)
0 – 1.4		SILT, some sand, some clay, stiff, medium plastic, moist, brown. organics/rootlets to 0.2 m.			23.7 24.0 22.5					
1.4 – 3.0		SAND, some silt, trace gravel, compact, well graded, fine to medium grained, damp, brown.			3.3 3.7					

MONITORING WELL: BH24-8
ELEV.: 100.7 m



NOTES:
1. Borehole open and dry Immediately After Drilling.
2. Recorded Monitoring Well Groundwater Level Dry on May 24, 2024.

P.MACHIBRODA ENGINEERING LTD.

BOREHOLE 24-9

DRAWING NUMBER: 21273-10

PROJECT: PROPOSED COMMERCIAL DEVELOPMENT

LOCATION: LAVOIE STREET, BEAUVAL, SK

NORTHING (m): 6114210 **EASTING (m):** 333489 **ELEVATION (m):** 99.5 **DATE DRILLED:** MAY 22/24

SAMPLE TYPE: CUTTINGS SPLIT SPOON SHELBY TUBE

WATER LEVELS
- After Drilling
- During Drilling

DEPTH (m)	DESCRIPTION
0 – 1.8	SILT, some clay, stiff, low to medium plastic, moist, brown.
1.8 – 4.3	GLACIAL TILL, sand and silt, trace clay, trace gravel, compact, well graded, fine to coarse grained, moist, brown, oxide stained.
4.3 – 7.8	GLACIAL TILL, silt, some sand, some clay, trace gravel, very stiff, low plastic, moist, brown, oxide stained. sandy, hard below 5.7 m.
7.8 – 10.5	SAND, some silt, very dense, well graded, fine to medium grained, damp, light brown.

SAMPLE TYPE	SPT (N) BLOWS/ 300 mm	WATER CONTENT (%)	LIQUID LIMIT (%)	PLASTIC LIMIT (%)	UNIT WEIGHT (kN/m³)	COMPRESSIVE STRENGTH (kPa)	POCKET PEN. (kg/cm²)
Cuttings		21.7					
Cuttings		24.7					
Cuttings		22.9					
Split Spoon	24	5.8					
Cuttings		11.2					2.0
Split Spoon	97	7.2			22.4		
Cuttings		7.8					
Split Spoon	50/125	4.5					
Cuttings		4.3					

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

SHEET 1 OF 1

06-12-2024 Y:\2024 Projects\21273 - Beauval 3 lot commercial development\Drafting\BH9-21273.bor

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.

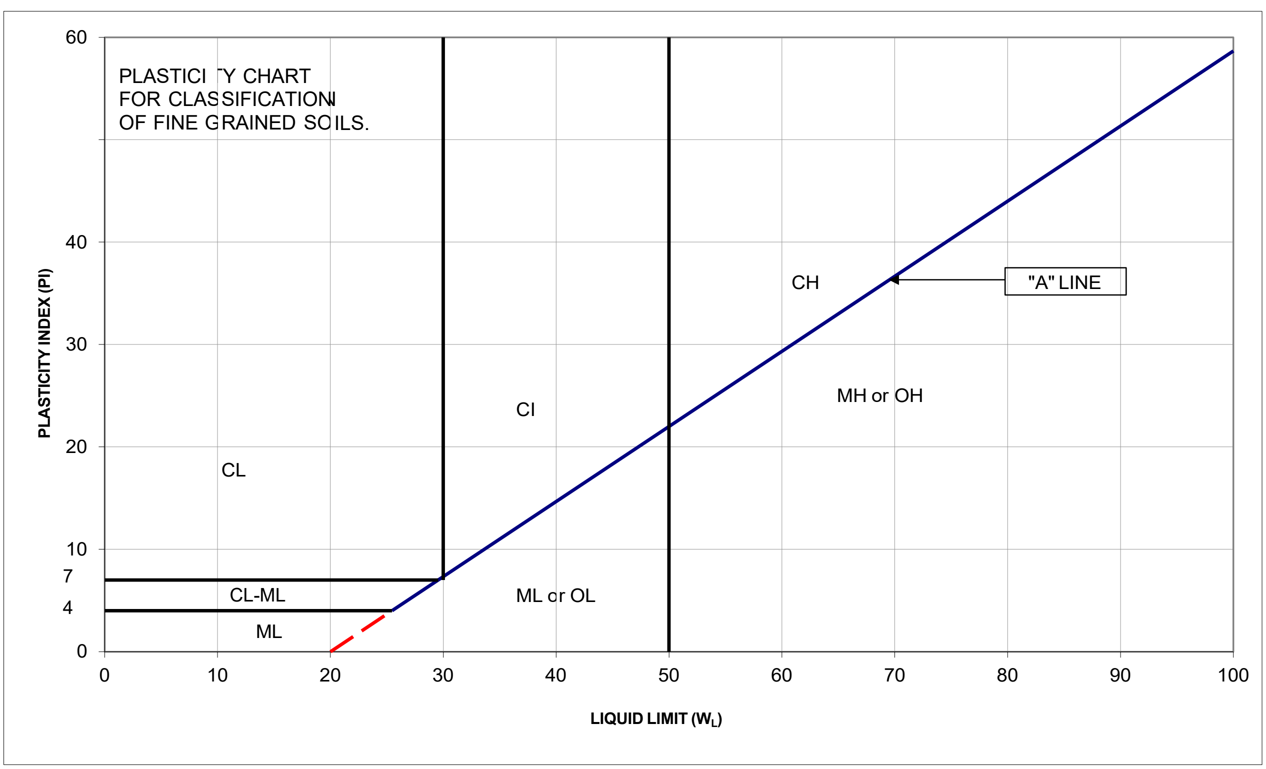
Organic Soils: Readily identified by colour, odour, spongy feel and frequently by fibrous texture.

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 Analysis
Test Results

P.MACHIBRODA ENGINEERING LTD.

AASHTO T 88: PARTICLE SIZE ANALYSIS OF SOILS

Project:	Proposed Commercial Development
Location:	Beauval, SK
Project No.:	21273
Date Tested:	May 30, 2024
Borehole No.:	24-1
Sample No.:	43
Depth (m):	1.5

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	100
# 10	2.000	99
# 20	0.850	98
# 40	0.425	97.1
#60	0.250	95.7
# 100	0.150	94.3
# 200	0.075	93.0

Hydrometer Analysis:

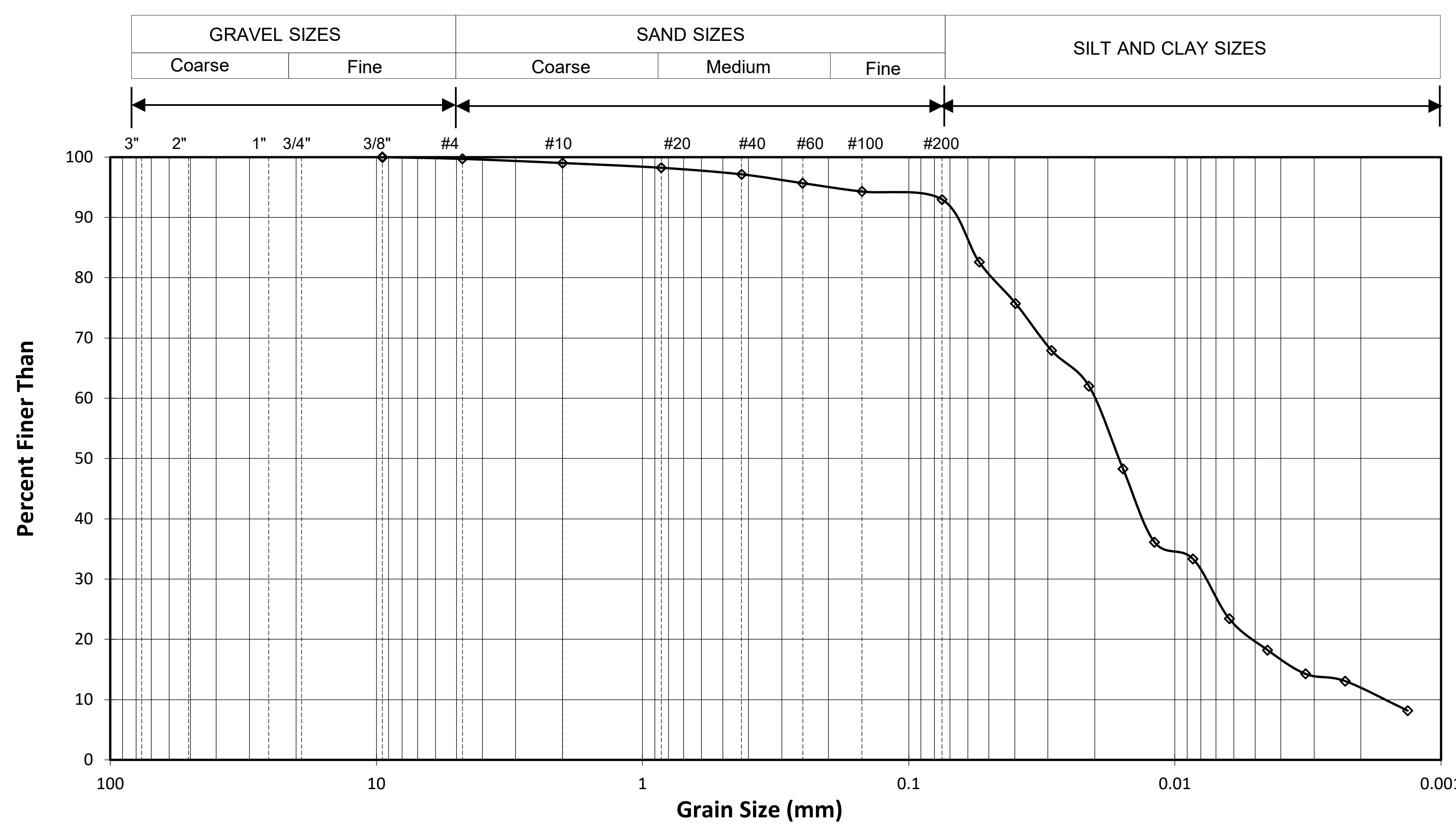
Dispersing Agent:
Sodium Hexametaphosphate

Diameter mm	% Finer
0.0543	82.6
0.0397	75.7
0.0290	67.9
0.0210	62.0
0.0157	48.3
0.0119	36.1
0.0085	33.3
0.0062	23.4
0.0045	18.2
0.0032	14.3
0.0023	13.1
0.0013	8.2

Material Description:

% Gravel Sizes	% Sand Sizes	% Silt Sizes	% Clay Sizes
0	0	88	12

Remarks:



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Drawing No.

Appendix B-1

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

P.MACHIBRODA ENGINEERING LTD.

ASTM C136: GRAIN SIZE ANALYSIS

Project: Proposed Commercial Development
Location: Beauval, SK
Project No.: 21273
Date Tested: May 31, 2024
Borehole No: 24-3
Sample No.: 32
Depth: 3.0

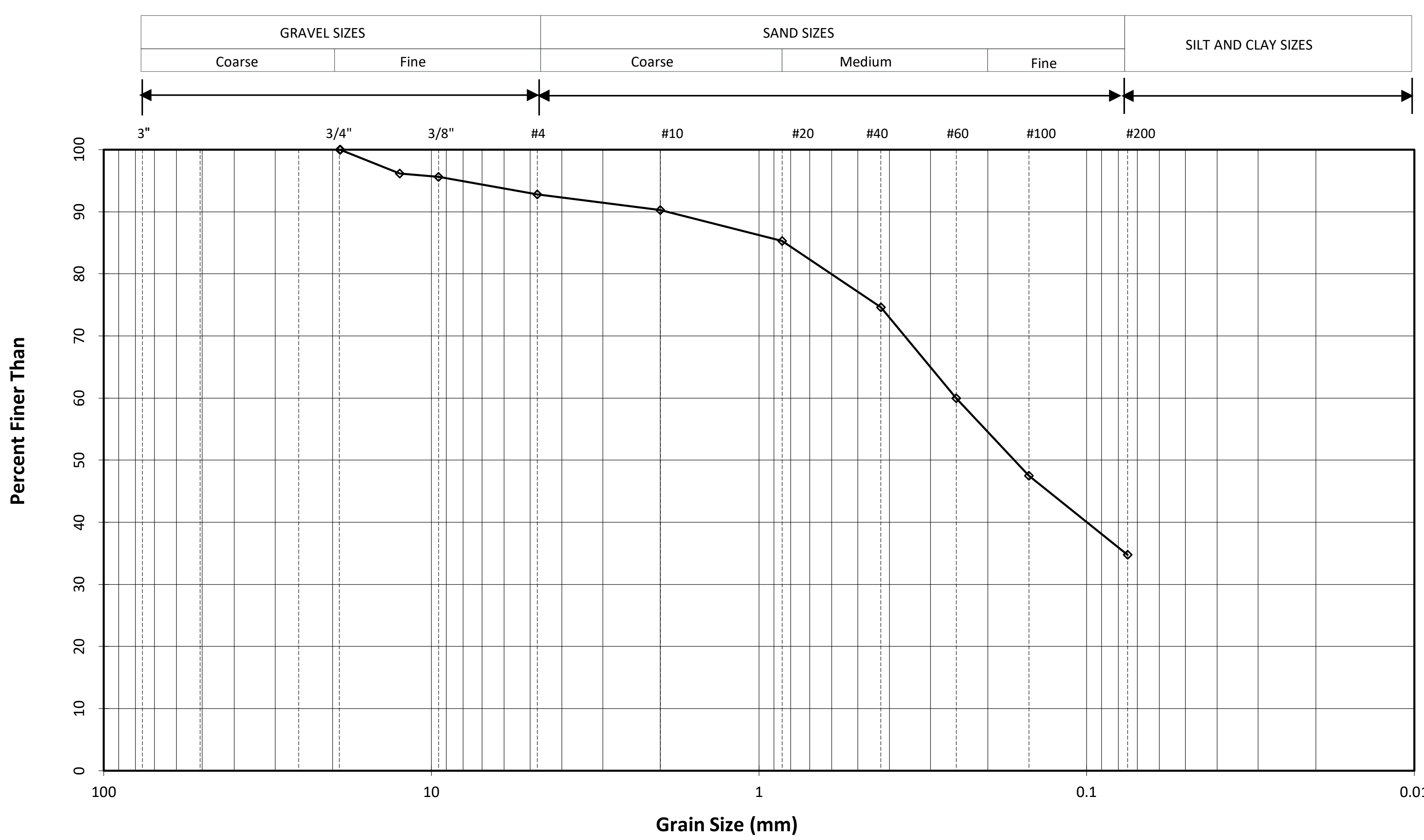
Sieve Analysis:

Sieve	Diameter mm	% Finer
	76.200	100
	63.500	100
	50.000	100
	37.500	100
	25.000	100
	19.000	100
	12.500	96
	9.500	96
	4.750	93
	2.000	90
	0.850	85
	0.425	75
	0.250	60
	0.150	47
	0.075	35

Material Description:

% Gravel Sizes	% Sand Sizes	% Silt and Clay Sizes
7	58	35

Remarks:



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DRAWING NO.

Appendix B-2

WE CERTIFY TESTING PROCEDURES ARE IN ACCORDANCE WITH ASTM C136 AND C117 CSTANDARDS
P. MACHIBRODA ENGINEERING LTD.

PER Preston Schergevitch

P.MACHIBRODA ENGINEERING LTD.

AASHTO T 88: PARTICLE SIZE ANALYSIS OF SOILS

Project:	Proposed Commercial Development
Location:	Beauval, SK
Project No.:	21273
Date Tested:	May 30, 2024
Borehole No.:	24-5
Sample No.:	51
Depth (m):	3.0

Sieve Analysis:

Sieve	Diameter mm	% Finer
1.5"	38.100	100
1"	25.400	100
3/4"	19.100	100
1/2"	12.700	95
3/8"	9.500	94
# 4	4.750	90
# 10	2.000	87
# 20	0.850	82
# 40	0.425	75.2
#60	0.250	66.8
# 100	0.150	59.9
# 200	0.075	53.2

Hydrometer Analysis:

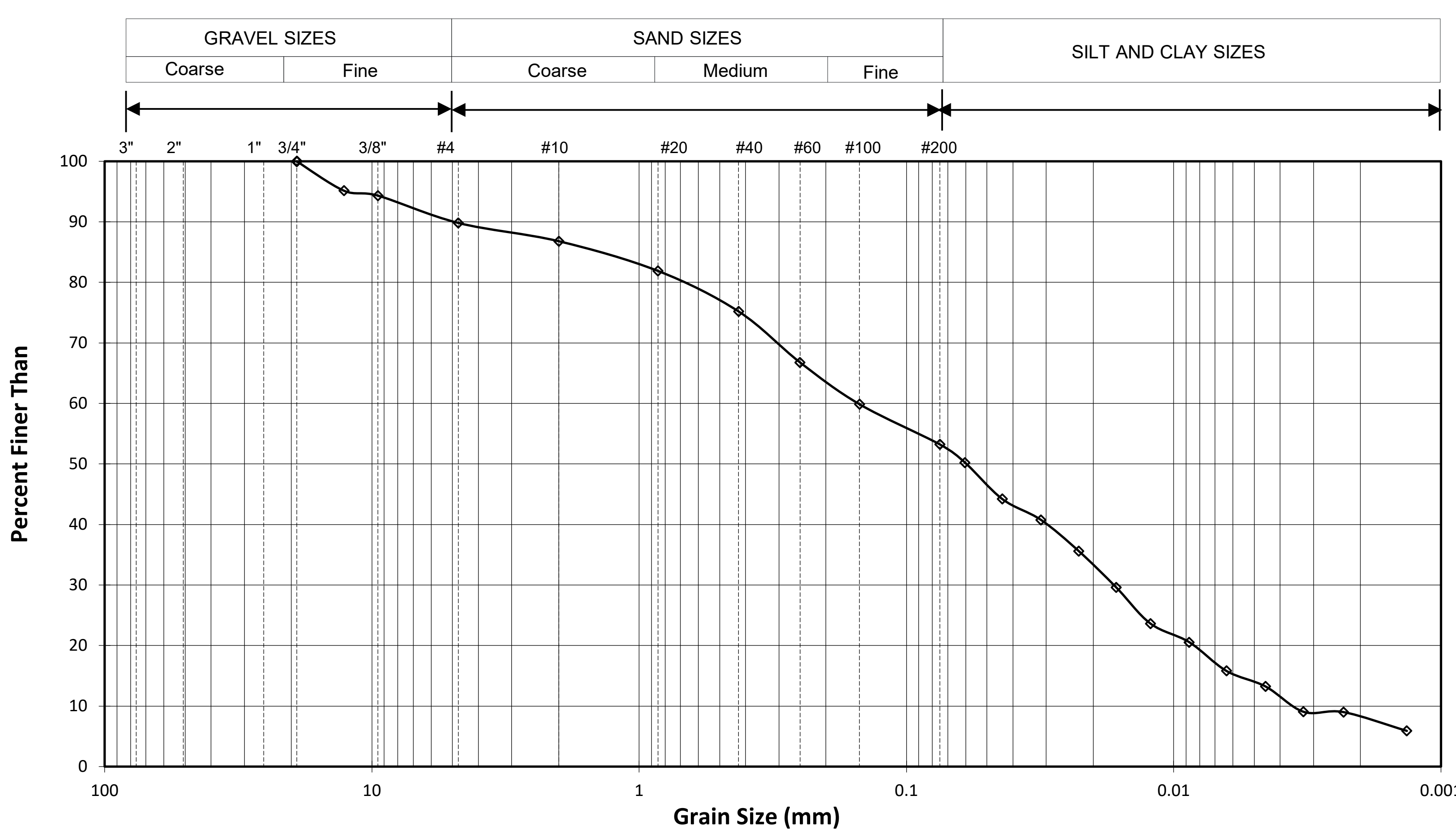
Dispersing Agent:
Sodium Hexametaphosphate

Diameter mm	% Finer
0.0604	50.2
0.0438	44.2
0.0314	40.8
0.0227	35.6
0.0164	29.6
0.0122	23.6
0.0087	20.6
0.0064	15.8
0.0045	13.2
0.0033	9.1
0.0023	9.0
0.0013	5.9

Material Description:

% Gravel Sizes	% Sand Sizes	% Silt Sizes	% Clay Sizes
10	37	45	8

Remarks:



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Drawing No.

Appendix B-3

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

PER Preston Schergevitch

P.MACHIBRODA ENGINEERING LTD.

ASTM C136: GRAIN SIZE ANALYSIS

Project: Proposed Commercial Development
Location: Beauval, SK
Project No.: 21273
Date Tested: May 31, 2024
Borehole No: 24-8
Sample No.: 13
Depth: 2.0

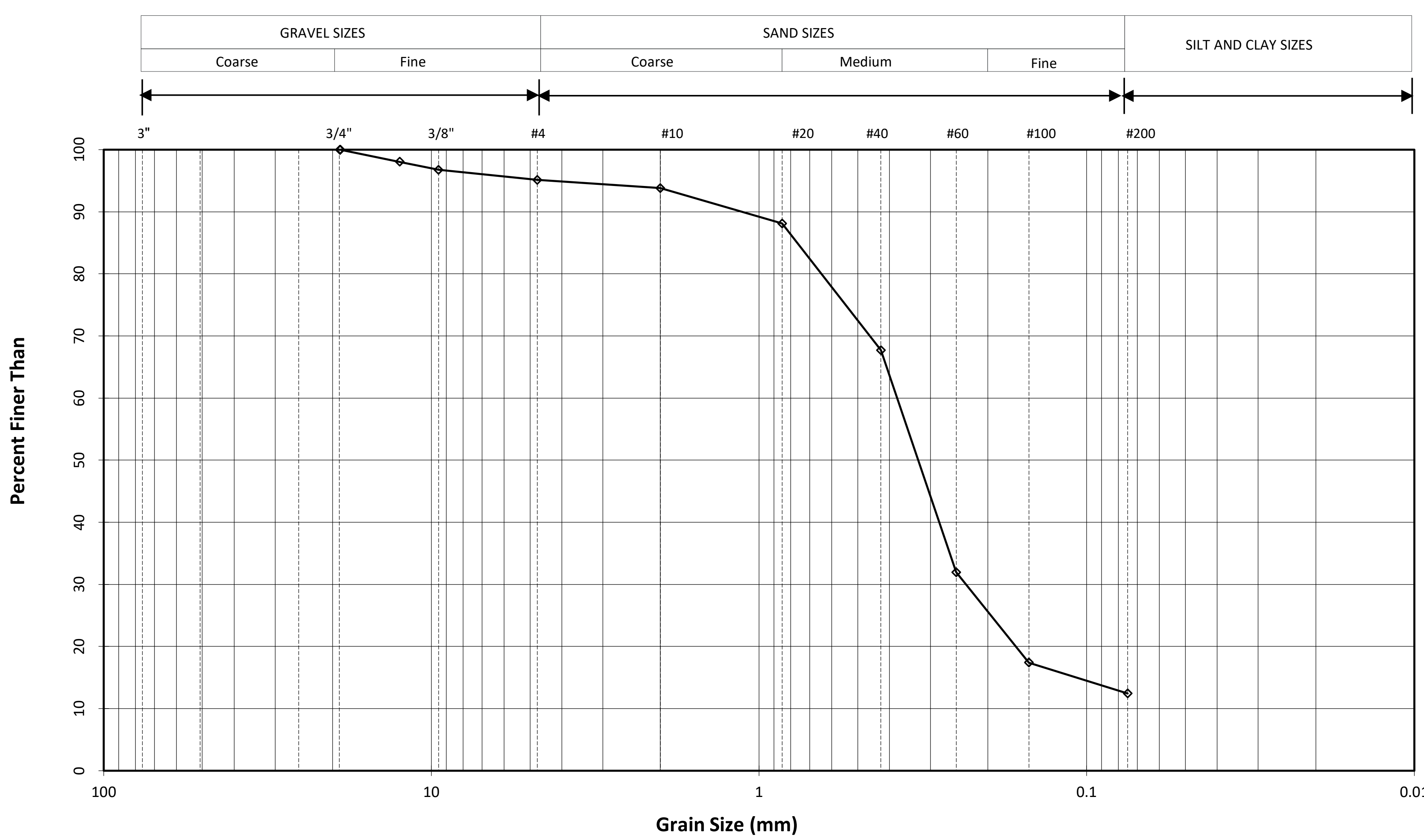
Sieve Analysis:

Sieve	Diameter mm	% Finer
	76.200	100
	63.500	100
	50.000	100
	37.500	100
	25.000	100
	19.000	100
	12.500	98
	9.500	97
	4.750	95
	2.000	94
	0.850	88
	0.425	68
	0.250	32
	0.150	17
	0.075	12

Material Description:

% Gravel Sizes	% Sand Sizes	% Silt and Clay Sizes
5	83	12

Remarks:



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DRAWING NO.

Appendix B-4

WE CERTIFY TESTING PROCEDURES ARE IN ACCORDANCE WITH ASTM C136 AND C117 CSTANDARDS
P. MACHIBRODA ENGINEERING LTD.

PER Preston Schergevitch