GEOTECHNICAL INVESTIGATION

PROPOSED SUBDIVISION
SUNSET BEACH RESORT – PHASE 2
LSD 7, 9 & 10-27-25-6-W3M
RM OF LOREBURN NO. 254, SK
PMEL FILE NO. 16158
OCTOBER 31, 2019

PREPARED FOR:
Prairie's Edge Development Corporation

ATTENTION: Ms. Lorraine Forster

P.MACHIBRODA
ENGINEERING LTD.
DOWN TO EARTH SOLUTIONS

PROJECT: Geotechnical Investigation
Proposed Subdivision
Sunset Beach Resort – Phase 2
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PREPARED FOR: Prairie's Edge Development Corporation
Box 3370
Humboldt, Saskatchewan
S0K 2A0

ATTENTION: Ms. Lorraine Forster

DISTRIBUTION: Prairie's Edge Development Corporation – One Copy
P. Machibroda Engineering Ltd. – One Copy

806-48th Street East, Saskatoon, SK S7K 3Y4
P: 306.665.8444 | **F**: 306.652.2092 | **E**: pmel.sk@machibroda.com
www.machibroda.com

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1 INTRODUCTION

1.1 GENERAL

The following report has been prepared on the subsurface soil conditions existing at the site of the proposed Subdivision – Prairie's Edge Resort Phase 2, to be constructed within LSD 7, 9 & 10-27-25-6-W3M, in the RM of Loreburn No. 254, Saskatchewan

The terms of reference for this investigation were presented in P. Machibroda Engineering Ltd. (PMEL) Proposal No. 16158, dated September 13, 2019. Written authorization to proceed with the investigation was provided via the signed Consulting Agreement between the Prairie's Edge Development Corporation and PMEL dated September 19, 2019.

PMEL had completed a geotechnical investigation for Phase 1 of the subdivision in 2017 (refer to PMEL Report No. 13147, dated November 6, 2017).

1.2 SITE LOCATION AND DESCRIPTION

The subject site is located along the east side of Lake Diefenbaker. Phase 2 of the subdivision will consist of new lots located generally along the lakeshore to the west and southwest of Phase 1 and along the road to the boat launch north of Phase 1.

The subject site was vacant land covered in grasses and few bushes/trees along the bank. In general, the subject site was flat and sloped gently towards the southwest (towards Lake Diefenbaker). Surficial slumping, due to erosion from wave action and ice forces was observed along the shoreline. The erosion was more prevalent within areas that the shoreline was exposed to Lake Diefenbaker in comparison to inlets further inland.

A Site Plan showing the location of the study area and test locations has been shown on Drawing Nos. 16158-1 and 1A.

2 FIELD INVESTIGATION

The field test drilling and soil sampling was conducted on October 3, 2019. Groundwater monitoring was conducted on October 17, 2019.

The ground surface elevation at the test locations were referenced to the top of found iron pins, located approximately as shown on the Site Plan, Drawing No. 16158-1. Test Hole Nos. 19-1, 19-2, 19-5 and 19-6 were referenced to Pin No. 35 (geodetic elevation of 563.963 m), and Test Hole Nos. 19-4 and 19-3 were referenced to Pin No. 3 (geodetic elevation of 559.350 m). The geodetic elevations for the found iron pins were provided by Meridian Surveys.

Six (6) test holes, located as shown on the Site Plan, Drawing No. 16158-1, were dry drilled using our truck-mounted, continuous flight auger drilling rig. The test holes were 150 mm in diameter and extended to a depth of 6 to 18 m below the existing ground surface.

Test hole drill logs, as shown on the attached Field Drill Logs, Drawing Nos. 16158-2 to 7, 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 (N-Index), utilizing a safety hammer with automatic trip were performed during test drilling.

Standpipe piezometers (50 mm PVC, machine slotted) were installed in Test Hole Nos. 19-1, 19-3 and 19-6 to monitor the groundwater levels.

3 SOIL AND GROUNDWATER CONDITIONS

3.1 SOIL PROFILE

The general soil profiles consisted of a thin layer of surficial topsoil overlying silt (to depths of 0.7 to 1.4 m below existing grade) followed by an extensive deposit of glacial till, which extended to a depth of at least 18.4 m below existing grade, the maximum depth explored with our testing at this site.

The silt was generally very stiff to hard in consistency and low to medium plastic. The glacial till was medium plastic and initially stiff to hard becoming hard below approximately 10 m.

3.2 GROUNDWATER CONDITIONS, SLOUGHING

The test holes remained dry and open during and immediately following test drilling. A summary of the groundwater levels recorded in the piezometers installed during this investigation has been presented in Table I.

TABLE I RECORDED GROUNDWATER LEVELS

Test Hole No.	Piezometer Rim Elevation (metres)	Ground Surface Elevation (metres)	Groundwater Depth (metres)		Groundwater Elevation (metres)	
			I.A.D.[1]	October 17, 2019	I.A.D.[1]	October 17, 2019
19-1	561.0	560.2	Dry	4.5	--	555.7
19-3	561.0	560.1	Dry	Dry	--	--
19-6	563.9	562.9	Dry	8.9	--	554.0

[1]I.A.D. – Immediately After Drilling

An examination of Table I revealed that the groundwater level was situated approximately 4.5 to 8.9 m below existing grade on October 17, 2019. Higher groundwater conditions could be encountered, particularly if the piezometers have not stabilized or following precipitation and/or spring thaw.

3.3 COBBLESTONES AND BOULDERS

Cobbles/boulders were encountered during drilling. The depths at which cobbles/boulders were encountered have been shown on the test hole logs.

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). Intertill/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 six (6), small diameter test holes drilled at this site was low. The frequency of encountering such deposits will increase proportionately with the number/depth of piles installed 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, 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 drill logs alongside the corresponding depths at which the samples were recovered, as shown on Drawing Nos. 16158-2 to 7, inclusive.

The results of the grain size distribution analyses have been presented in Appendix B.

5 SLOPE STABILITY

The theoretical slope stability analysis was performed using the SLOPE/W computer program available through Geo-Slope International Ltd.[1] The Morgenstern-Price Method of slices was used for all analysis (utilizing a half-sine side force function). The slope was analyzed for circular failure and composite failures (i.e., translational). The purpose of the slope stability analysis was to provide the recommended building setback for structures (i.e., houses, garages, etc.) that may be built on the proposed lots.

5.1 BACKGROUND

PMEL conducted a slope stability study of the subject site for Phase 1 of the subdivision in 2017 (refer to PMEL Report No. 13147, dated November 6, 2017, "2017 Geotechnical Report"). At the time of the current investigation Phase 1 and the boat launch had been developed, and the area of Phase 2 was vacant pastureland covered with grasses with some trees. Outside the above noted recent development on the subject site, there were no significant changes within the area of Phase 2 since 2017.

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[1] Geo-Slope International Ltd., 2012. Slope/W User's Manual, A Comprehensive Program for Slope Stability Analysis, Geo-Slope International Ltd., Calgary, Alberta.

As noted in the 2017 Geotechnical Report, the subject site was flat and sloped gently towards the southwest (towards Lake Diefenbaker). There were no signs (i.e., tension cracking, land not terraced, etc.) of a deep-seated landslide (historical or current) impacting the subject site. However, some surficial slumping, due to erosion from wave action and ice forces was observed along the shoreline. The erosion was more prevalent within areas that the shoreline was exposed to Lake Diefenbaker in comparison to inlets further inland.

It is difficult to estimate magnitude of erosion over the years, but based on historical aerial photographs/imagery, the shoreline does not appear to have significantly altered between 1988 and present day. Though some slight erosion was evident along the west shore between 2013 and 2019 (as based on aerial imagery).

5.2 INPUT FOR ANALYSIS

The surface geometry of the subject site was interpreted from a topographical survey provided by Associated Engineering. Stability analysis was conducted at three locations along the shoreline, Stratigraphic Sections AA-AA', BB-BB' and CC-CC', located approximately as shown on Drawing No. 16158-1.

The stratigraphic units as well as the lithologic boundaries were interpreted from the results of the subsurface soils investigation.

The piezometric conditions used for the slope stability analysis were interpreted from the recorded water levels in the installed piezometers, and the approximate water elevation of Lake Diefenbaker. A hydrostatic pore pressure condition was used for the analysis.

A sensitivity analysis was performed to assess changes in the calculated Factor of Safety as a result of potential variations in the static groundwater levels and lake levels. Based on historical water levels within Lake Diefenbaker the lake levels can vary up to 7 m through the year (has varied approximately 5 m as of October 29, 2019)[2].

The soil properties obtained during this investigation as well as the design strength parameters used for the theoretical slope stability analysis have been presented in Table II. The soil strength parameters selected for analysis were based on published strength parameters and laboratory testing on soil samples collected during this investigation.

TABLE II SLOPE STABILITY SOIL PARAMETERS

Material Type	Total Unit Weight (kN/m^3)	Effective Unit Cohesion (kPa)	Effective Internal Angle of Friction (Degrees)
Glacial Till	21.5	5	28

[2] Water Security Agency. *Lake Diefenbaker at Gardiner Dam*. https://www.wsask.ca/Lakes-and-Rivers/Stream-Flows-and-Lake-Levels/South-Saskatchewan-River-Watershed-/05HF003/. Accessed October 29, 2019.

5.3 Results of Analysis

The results of the slope stability analysis have been presented below in Table III. Typical plots have been presented in Appendix C.

Table III Slope Stability Analysis Results

Slope Section	Piezometric Condition	Calculated Factor of Safety	Drawing No.
AA-AA’	Inferred	2.5	C-1
BB-BB’	Inferred	2.5	C-2
CC-CC’	Inferred	2.4	C-3
CC-CC’	5 metre drop in lake water level	1.9	C-4

Based on the slope stability analyses the Factor of Safety at the back property line (lakeside) of the proposed lots was 2.5 for Slope Sections AA-AA’ and BB-BB’, and 2.4 for Section CC-CC’.

Based on the sensitivity analysis the most significant impact to slope stability would be a sudden drop in the lake water level. Based on a 5 m drop in water level the Factor of Safety at Slope Section CC-CC’ dropped from 2.4 to 1.9.

5.4 Discussion/Slope Stability Considerations

The Factor of Safety of a slope is defined as the ratio of the available shear strength of the soil, to the minimum shear strength required to maintain stability. A Factor of Safety of less than or equal to 1.0 would indicate the potential for slope failure. A minimum Factor of Safety that is considered acceptable for most permanent structures (such as a house) constructed adjacent or on a slope is 1.5 (source: Canadian Foundation Engineering Manual, 2006). As such, a minimum Factor of Safety of 1.5 is recommended for the building setback from the crest of slope for long term stability.

Based on the slope stability analysis, the Factor of Safety of the slopes was at least 2.4, which exceeded the minimum recommended Factor of Safety of 1.5. As such, there is no recommended geotechnical building setback at this site provided any permanent structures are built within the proposed lot boundaries. A sudden drop in the lake level reduced the Factor of Safety by approximately 20 percent, but the Factor of Safety was still above 1.5 which indicates that the slope would remain stable.

The biggest risk to the proposed lots appears to be shoreline erosion due to wave action and ice forces. Based on the aerial photograph review there did not appear to have been significant shoreline degradation within the area of the subject site since at least 1988, but erosion and surficial slumping is still occurring. Surficial slumping and erosion along the shoreline was most prevalent in the area of proposed Block 1, Lot Nos. 23 to 29 and 40 to 47, inclusive (refer to Drawing No. 16158-1A). As such, it is recommended that the shoreline be carefully monitored for signs of slumping/erosion and if it appears to be encroaching on the proposed lots, shoreline erosion protection measures should be implemented.

Overall the existing slopes are considered stable and based on the slope stability there are no recommended geotechnical building setback restrictions within the proposed lot boundaries. The construction of the proposed Subdivision should not significantly impact the overall stability of the slope provided the following precautions and recommendations are followed.

- Irrigation of lawns, trees, shrubs, etc., should be kept to a minimum; permanent sprinkler and/or irrigation systems shall not be constructed;
- Drainage and/or discharge of water (i.e., roof downspouts, sump pump discharges, etc.) should not be channeled over the slope and should be directed towards the front of the house away from the slope;
- Existing drainage paths should not be altered;
- The lots should be graded and/or landscaped to ensure there is no ponding or runoff of water over the slope. Runoff should be directed towards the front of the lots;
- Existing vegetation should be disturbed as little as possible. Where vegetation is disturbed, erosion control and re-vegetation of the area should be implemented immediately;
- Construction activities (i.e., dumping of fill, construction of pathways, etc.) should not encroach on the existing slope. Site grading (i.e., fill placement) at the proposed house location should be kept to a minimum and should comply with the approved lot grading plan; and
- A septic field should not be constructed at this site. Residential sewage should be discharged into a CSA approved holding tank and disposed off-site. For Block 1; Lot Nos. 23 to 29 and 40 to 55, inclusive, the holding tanks should be located on the front half of the lots (road side). There are no site restrictions for the holding tanks installed for the rest of the Lots in Phase 2.

In addition to the above, the slope should be monitored carefully for signs of instability and conditions that could negatively impact stability. Signs of instability include tension cracks (i.e., cracks in the ground on the slope), differential movement of foundations, leaning trees and slumping.

Conditions that could negatively impact stability include ponding of water on the slope, erosion on and at the toe of slope, loss of vegetation on slope and significant changes in the lake levels. If any of these signs and conditions are observed, the Geotechnical Consultant should be contacted immediately to reassess our analysis and provide remedial options, if applicable.

6 GEOTECHNICAL DESIGN RECOMMENDATIONS

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

6.1 DESIGN CONSIDERATIONS

The subsurface soil conditions consist of a thin layer of organic topsoil overlying silt followed by glacial till.

The groundwater level was measured at 4.5 to 8.9 m below existing grade on October 17, 2019. Higher groundwater conditions could be encountered, particularly if the piezometers have not stabilized or following precipitation and/or spring thaw.

The potential depth of frost penetration for the soils at this site could range from approximately 1.5 to 2.5 m, depending on surface cover and severity of winter; the depth of frost penetration will be greater where granular fills/soils are present. Buried utilities should be based below the depth of frost penetration or protected against frost action with strategically placed insulation (PMEL can provide insulation recommendations upon request).

A shallow footing foundation based below the average depth of frost penetration on undisturbed, naturally deposited soil should perform satisfactorily as foundation support for the proposed homes. Alternately, a deep pile foundation consisting of drilled, cast-in-place concrete piles or helical screw piles could be utilized and should perform satisfactorily.

Recommendations have been prepared for site preparation; excavations; site classification for seismic site response; limit states resistance factors and serviceability; footings; deep foundations; floor slabs; foundation walls; foundation concrete and roadway structures.

6.2 SITE PREPARATION

All organic topsoil, loose fill and deleterious materials should be removed from the construction area. Staining and root intrusion from the 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. Additional information regarding topsoil composition and soil structure is presented in Appendix D.

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 below specified densities. 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.

The following minimum density requirements are recommended for this site.

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.

Fill, required to bring the subgrade surface to the design elevation in construction areas, should preferably consist of imported granular material or approved non-expansive, fine grained soils.

All proposed subgrade fill 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 96 percent of standard Proctor density at optimum moisture content.

Utility trench excavations are susceptible to settlement and should be adequately backfilled and compacted. The magnitude of settlement is directly related to the level of compaction of the backfill material. Well compacted fills will settle a small percentage of the fill thickness whereas poorly compacted fills can settle appreciably, particularly if frozen soils are incorporated in the backfill. Efforts should be made to meet the specified compaction level in areas sensitive to settlement.

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

6.3 EXCAVATIONS AND DEWATERING

Temporary excavations should be designed and excavated in accordance with current Saskatchewan Occupational Health and Safety Regulations. The Contractor is solely responsible for protecting the excavation by shoring, sloping, benching and/or other means as required to maintain the stability of both the excavation sides and the bottom.

Within the proposed depth of excavation at this site, the silt and glacial till deposits may be classified as “Type 2 to 3” soils. Sideslopes should be no steeper than 1H:1V, as measured from the bottom of the excavation. Slope flattening will be required if unstable conditions are encountered during excavation. Continuous visual monitoring of the sideslopes should be undertaken to assess whether flatter sideslopes are required to maintain stability.

The groundwater level was measured at 4.5 to 8.9 m below existing grade on October 17, 2019. Higher water levels should be expected during or following spring snowmelt and/or during or following periods of precipitation.

De-watering should be conducted on an “as required” basis over the time period for which the excavations are left open. A sump (or multiple sumps, if required) should be set up at the deepest excavation points and the floor of the excavation sloped to the sump(s) to handle groundwater seepage and precipitation runoff. A self-actuated sump pump(s) should be operated on a continuous basis and should be discharged well away from the excavations.

Depending on lateral constraints, excavations at this site may be completed with unbraced, sloped side walls. If there is insufficient room for excavation cuts, due to close proximity to other structures, then a temporary shoring system would be required.

6.4 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 2015 National Building Code, the site classification for seismic site response falls within Class D.

6.5 LIMIT STATES RESISTANCE FACTORS AND SERVICEABILITY

The National Building Code of Canada (NBCC, 2015) 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 (2015) and/or the Canadian Foundation Engineering Manual (CFEM, 2006).

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 (2015), 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, $\Phi = 0.3$

Shallow foundations:

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

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 6.6 and 6.7.3.

6.6 FOOTINGS

A shallow footing foundation based below the depth of frost penetration on naturally deposited, undisturbed soil should perform satisfactorily at this site.

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. 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 soil.

2. For permanently heated, at-grade structures where heat loss to the foundation is permitted, the footings should be based at a minimum depth of 1.5 m below finished grade. 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., 2.0 m) or protected against frost action with strategically placed insulation (PMEL can provide insulation recommendations upon request if shallower foundation depths than recommended are desirable).

3. If insulation is utilized beneath the basement floor slab, it is recommended to leave a one (1) m gap of uninsulated space along the perimeter of the floor to allow heat loss to the underside of the footings. The footings should extend deeper if the entire slab areas is insulated.

4. Footings based on naturally deposited, undisturbed soil may be designed to exert a ULS bearing pressure of 600 kPa. The SLS bearing pressure to limit settlements to less than 25 mm would be 150 kPa. 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.

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.

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.

8. The footing excavation should be hand-cleaned to remove all loose, disturbed soil, and to expose naturally deposited, undisturbed soil.

9. 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.

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

6.7 DEEP FOUNDATIONS

6.7.1 DRILLED, CAST-IN-PLACE CONCRETE PILES

Some construction difficulties associated with cobbles/boulders and saturated granular deposits should be anticipated. Temporary casing may be required to complete the installation of conventional drilled piles at this site.

Drilled, cast-in-place concrete, straight shaft 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 IV SHAFT RESISTANCE (DRILLED PILES)

Depth (metres) [1]	Shaft Resistance (kPa)	
	Unfactored ULS	SLS
0 to 2	0	0
Below 2	75	30

[1] Depth below existing ground level.

Notes:

1. To minimize frost heave potential, drilled piles exposed to frost action should be extended to a minimum depth of 6 m below finished ground surface. Lightly loaded exterior piles may need to be extended deeper to reduce the potential for frost heaving. If applicable, PMEL should be notified to reassess the minimum installation depth.
2. Piles should be reinforced to withstand all axial and lateral forces within the pile.
3. 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.
4. The pile holes should be filled with concrete as soon as practical after drilling.
5. 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.
6. A minimum centre-to-centre pile spacing of not less than three pile diameters is recommended.
7. Construction difficulties associated with cobbles/boulders may be encountered. Coring and/or preboring may be required at some locations.
8. A representative of the Geotechnical Consultant should inspect and document the installation of the drilled, cast-in-place concrete piles.

6.7.2 Helical Screw Piles

Helical screw piles are installed by rotating a steel pipe, equipped with one or more helix flightings, into the ground. For single helix screw piles, pile capacity is derived from shearing resistance along the pile shaft (i.e., shaft resistance) as well as end bearing capacity of the helix.

For multi-helix piles, pile capacity may be derived from the sum of the shearing resistance along the portion of pile shaft above the uppermost helix and end bearing capacity of each helix. The helical plates should be spaced a minimum of 3 helix diameters apart.

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

Table V Shaft Resistance (Screw Piles)

Depth (metres) [1]	Shaft Resistance (kPa)	
	Unfactored ULS	SLS
0 to 2	0	0
Below 2	45	18

[1] Depth below existing ground level.

Table VI End Bearing Resistance (Screw Piles)

Depth (metres) [1]	End Bearing Resistance (kPa)	
	Unfactored ULS	SLS
Below 3	1,000	400

[1] Depth below existing ground level. Torque monitoring must be conducted to confirm piles are based in suitable bearing strata / to confirm that soil conditions are as expected.

Notes:

1. The minimum embedment depth of the uppermost helix for multi-helix piles should be ≥ 3 m or H/D = 5 (whichever is greater), where H = depth to top helix, D = helix diameter.
2. Single helix screw piles should extend to a minimum depth of 5 m below grade or H/D = 5 (whichever is greater).
3. Lightly loaded exterior piles may need to be extended deeper to reduce the potential for frost heaving. If applicable, PMEL should be notified to reassess the minimum installation depth
4. When determining the compressive shaft resistance of the pile shaft, the portion of the pile shaft within 1D above the uppermost 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 uppermost helix.
5. Compressive end bearing capacity may be calculated utilizing the effective soil contact area of the helix (i.e., overall cross-sectional area for the lowest helix, helix area minus shaft area for upper helixes). 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.

6. A minimum centre-to-centre pile spacing of 2.5D is recommended, where D=helix diameter.

7. 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.

8. 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.

9. 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.

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

6.7.3 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 anticipated settlement of individual straight shaft piles and enlarged base piles (i.e., screw piles) is in the order of 5 to 10 mm and 10 to 20 mm, respectively. 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.

6.7.4 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 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.

6.8 Floor Slabs

6.8.1 Design Considerations

Grade-supported slabs (i.e., basement and garage floor slabs, exterior slabs, etc.) based on the silt and/or glacial till subgrade soils should perform satisfactorily.

6.8.2 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 grade beam to direct heat to the underside of the slab. The insulation should have a minimum thickness of 75 mm and should extend sub-horizontally to a minimum distance of 1.8 m beyond the outer edges of the slab. If differential movements cannot be tolerated, the slab could be structurally supported on piles.

6.8.3 BASEMENT FLOOR SLAB

Provided some potential differential floor movements are considered acceptable, the following minimum provisions have been provided to assist in the design of a conventional, heated, grade-supported, cast-in-place, reinforced concrete slab subject to light loading.

1. Prepare the site in accordance with Section 6.2, Site Preparation. Excavate soft subgrade areas and replace with suitable subgrade fill. The need for special measures (i.e., geotextile/geogrid placement) should be subject to review by the Geotechnical Consultant.

2. Subgrade fill, if required, should preferably consist of granular material or non-expansive (i.e., low plastic) fine-grained soils, placed in thin lifts (maximum 150 mm loose) and compacted to 96 percent of standard Proctor density at optimum moisture content.

3. Provide a minimum of 200 mm of clean, drainage aggregate below the floor slab. Shape the subgrade surface to allow for free drainage to a sump pit(s). The drainage aggregate should meet the following gradation requirements.

TABLE VII CLEAN, DRAINAGE AGGREGATE

Grain Size (mm)	Percent Passing
25.9	100
9.5	50 – 95
5.0	35 – 70
2.0	20 – 45
0.425	0 – 20
0.150	0 – 8
0.071	0 – 3

4. The granular fill should be placed in thin lifts (150 mm loose) and compacted to 98 percent of standard Proctor density at optimum moisture content.

5. 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.

6. A sump pit is recommended below the basement floor slab to collect any free water which may accumulate beneath the floor, and to collect water from the perimeter drainage system. The sump pit should be perforated to permit collection of water from the sub-slab granular fill and wrapped with a separation geotextile to prevent clogging. The sump pit should be equipped with an automatic sump pump.

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

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

9. Provide positive site drainage away from the proposed Residence. Extend downspouts and sump pump discharge outlets at least 3 m away from the foundation.

10. Floor slabs should not be constructed on desiccated, wet, or frozen subgrade soil or granular fill.

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

6.8.4 GARAGE FLOOR AND EXTERIOR SLABS

Design of the garage floor and exterior slabs should follow the general recommendations presented above. Slabs exposed to freezing conditions will undergo differential movements associated with frost action. Increasing the depth of non-frost susceptible granular fill or placement of rigid polystyrene insulation could be considered to help minimize the effects of frost action on exterior slabs. Such grade-supported slabs should also be free floating and not rigidly connected to the proposed residence.

6.9 FOUNDATION WALLS

Soil retaining walls should be designed to resist lateral earth pressure exerted by the soil as well as the horizontal pressure induced by any surcharge loading. The surcharge loading should be calculated on the basis of actual loads. Backfill placed against the wall should be uniformly placed and compacted to minimize settlements as much as practical and avoid development of compaction induced pressures on the wall. The equivalent fluid pressure distribution will be dependent upon the soil utilized as backfill around the foundation wall and should be as follows:

- Where the existing soils are used to backfill the foundation walls, the lateral earth pressure may be calculated on the basis of an equivalent fluid pressure distribution of 16 kN/m^3.
- Where clean granular fill (i.e., less than 5 percent material finer than 0.071 mm) is used to backfill the foundation walls, the lateral earth pressure may be calculated on the basis of an equivalent fluid pressure distribution of 10 kN/m^3.

To prevent hydrostatic pressures from developing behind the wall, a drainage system should be incorporated into the design of the wall. A perforated drainage pipe should be installed with the invert elevation at or below the base of the foundation. The perimeter drainage system should be drained to a sump pit. The sump pit should be equipped with an automatic sump pump.

The perforated drainage pipe should be at least 100 mm in diameter and installed on non-woven geotextile capable of transmitting a flow of not less than 50 litres per second per square metre (ASTM D-4491). The geotextile should be placed on naturally deposited, undisturbed soil or free-draining sand, as may be required for leveling. The geotextile should be used to encapsulate at least 300 mm of clean, granular drainage aggregate above the invert of the drainage pipe.

The clean drainage aggregate should meet the aggregate gradation requirements shown in Section 6.8.3, Table VII. All water collected in the drainage system must be discharged in accordance with local regulations.

The uppermost 500 mm of the backfill should consist of clay or other low permeability material.

6.10 FOUNDATION CONCRETE

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

TABLE VIII WATER SOLUBLE SULPHATE TEST RESULTS

Test Hole No.	Depth (metres)	Soil Type	Water Soluble Sulphate (%)	Class of Exposure	Degree of Sulphate Exposure
19-3	1.5	Till	0.046	--	Negligible
19-3	4.5	Till	0.69	S-2	Severe

An examination of Table VIII revealed that the measured sulphate concentration of the tested soils was 0.046 and 0.69 percent, which is considered negligible to severe in terms of potential degree of sulphate attack. As such, 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.

6.11 GRAVELLED ROADWAYS

6.11.1 DESIGN INPUTS

It is understood that the proposed roadway for Phase 2 will be a relatively low volume gravelled surfaced roadway. The provided rural low volume roadway recommendations have been based on average annual daily traffic (AADT) of less than 200 vehicles per day or predominately light duty traffic. Exceeding 200 AADT for the rural low volume roadways will result in increased maintenance and shortening of the roadways life span.

6.11.2 CONSTRUCTION

The following minimum recommendations should be incorporated into the design of gravelled roadways.

1. Prepare the subgrade in accordance with Section 6.2, Site Preparation. Typical subgrade preparation techniques (i.e., moisture conditioning and re-working the soil) should be applicable in most areas of the site. However, it is understood that there are low lying/wet/soft areas that may be built up/require additional treatment. As such, over-excavation/replacement of soft areas and/or reinforcing with geotextile/geogrid will likely be required in some areas during construction of the proposed roadway. The need for special provisions (i.e., over-excavation, geotextile, etc.) must be subject to review by the Geotechnical Consultant during field construction. Based on the actual conditions at the time of construction, specific construction recommendations relevant to the conditions observed can be provided. The road structure may need to be modified to accommodate the construction equipment and the intended use.

2. Based on the design inputs, the recommended gravel structures have been presented in Table IX.

TABLE IX RURAL – LOW VOLUME TRAFFIC STRUCTURES

Pavement Structure	Option 1	Option 2
Traffic Gravel	50[1]	--
Clay Cap	150	--
Granular Base (Min CBR = 65)	-	100
Geotextile/Geogrid[2]	-	-
Prepared Subgrade	(150)	(150)
Total Thickness (mm)	**200**	**100**

[1]The traffic gravel should be placed in two, 25 mm lifts with the first lift placed during construction and the second lift placed between years 1 and 2.

[2]High strength Geogrid/Geotextile (e.g., Combigrid 40/40, Mirafi HP270, Geotex 2x2HF or equivalent) will be required where soft subgrade soils are encountered. Prior to placement the Geotechnical Consultant should review the field conditions. Based on the field conditions, the roadway structure may need to be modified.

3. All granular fill placed above the 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 base course and traffic gravel material should meet the aggregate gradation requirements presented in Table X.

TABLE X AGGREGATE GRADATION REQUIREMENTS

Grain Size (mm)	Percent Passing[1]	
	Type 31 Base-Course	Type 109 Traffic Gravel
40.0	--	100
31.5	100	--
18.0	75 – 90	--
12.5	65 – 83	--
5.0	40 – 69	45 - 80
2.0	26 – 47	25 - 60
0.900	17 – 32	--
0.400	12 – 22	--
0.160	7 – 14	--
0.071	6 – 11	0 – 30
Plasticity Index (%)	0 – 6	4 - 12
CBR (Min)	65	25
% Fracture (Min)	50	50

[1]SK MHI Specifications

4. The finished surface should be graded with a centre crown and a minimum gradient of 3% to reduce the potential for moisture infiltration through the road structure.

5. Ditches and culverts should be provided where necessary to provide adequate site drainage. Ditch sideslopes should be no steeper than 3H:1V. The invert of the ditch should preferably be in the order of 1 m below the edge of the roadway to minimize the accumulation of snow during the winter months and maintain a freeboard above standing water in the ditch. A lesser depth (minimum of 500 mm) could be accepted in areas to satisfy lateral constraints.

6. Erosion protection is recommended for all embankment sideslopes. The slopes should be covered with topsoil and seeded to encourage vegetation growth. Alternately, erosion control blankets or hydromulch could be installed. Ditch blocks should be installed within the ditches to minimize soil erosion.

7. Periodic maintenance such as surface grading will be required to maintain the desired riding surface.

7 LIMITATIONS

The presentation of the summary of the field drill logs and foundation design recommendations has been completed as authorized. Six (6), 150 mm diameter test holes were dry drilled using our continuous flight, solid stem auger drilling equipment. Field drill logs were compiled for the Test Holes during test drilling which, we believe, were representative of the subsurface conditions at the Test Hole locations at the time of test drilling.

Variations in the subsurface conditions from that shown on the drill 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 geotechnical investigation did not include any environmental assessment of the site. No detectable evidence of environmentally sensitive materials such as hydrocarbon odour 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 test holes should be drilled and samples recovered for chemical analysis.

The subsurface investigation necessitated the drilling of deep test holes. The test holes 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 Test Hole location is maintained level with the existing grade.

This report has been prepared for the exclusive use of the Prairie's Edge Development Corporation and their agents for specific application to the proposed Subdivision – Prairie's Edge Resort Phase 2, to be constructed within LSD 7, 9 & 10-27-25-6-W3M, in the RM of Loreburn No. 254, Saskatchewan. It has been prepared in accordance with generally accepted geotechnical engineering practices and no other warranty, express or implied, is made.

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.

The acceptance of responsibility for the design/construction recommendations presented in this report with respect to the foundation system are contingent on adequate and/or full-time inspection (as required, based on site conditions at the time of construction) by a representative of the Geotechnical Consultant. 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 originally signed reports and 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
G.A.A. BAXTER
MEMBER 15402
2019 10 31
YR. MN. DAY
SASKATCHEWAN

Graham Baxter, P. Eng.

K. Pardoski

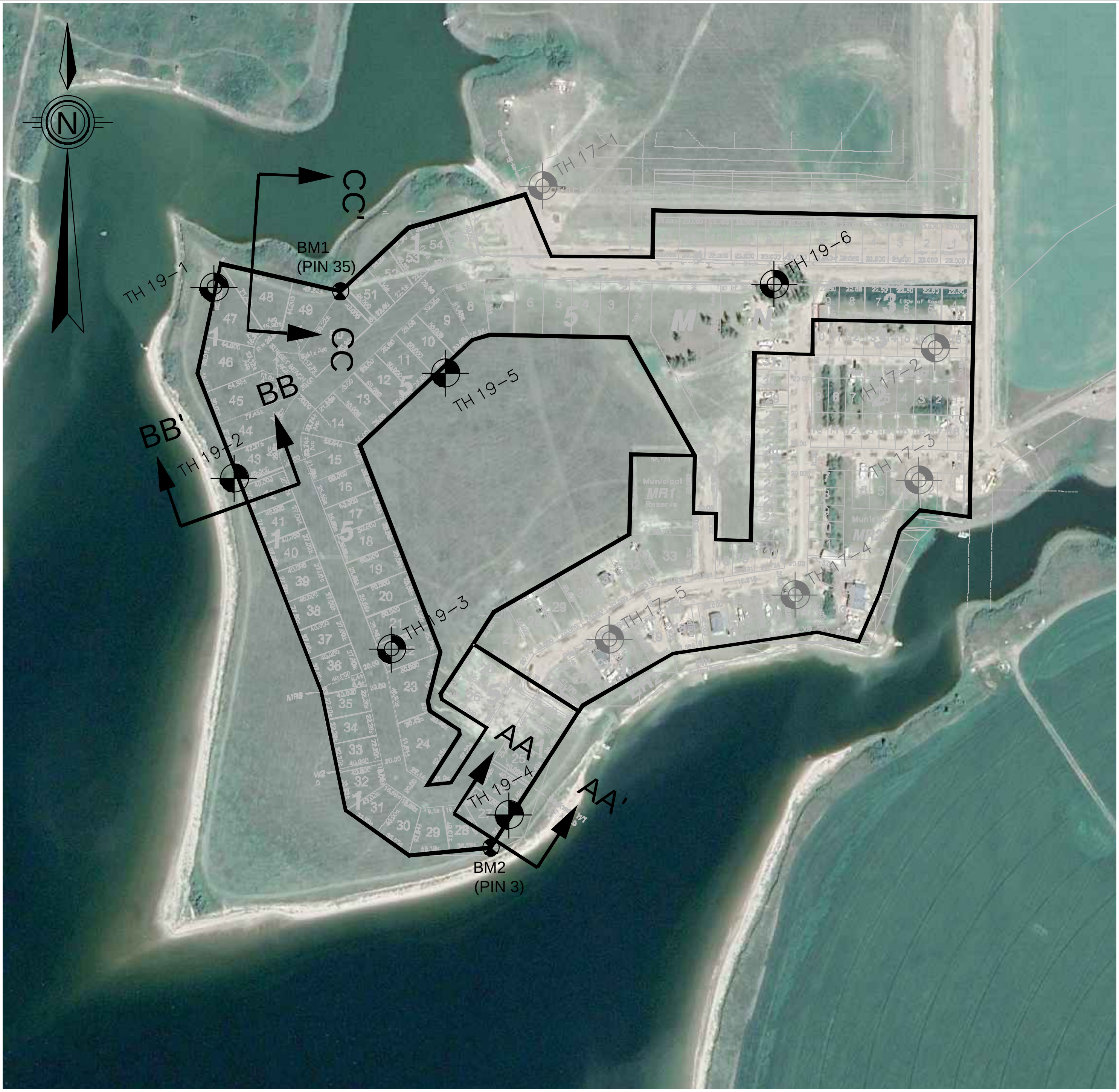
Kelly Pardoski, P. Eng.

GB:KP:zz

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	15402	2019-10-31

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 ©2019, IMAGE ©2019 DIGITALGLOBE, (IMAGERY DATE: 7/26/19).
3. BM1 - PIN No 35, GEODETIC ELEVATION = 563.963 m; BM2 - PIN No. 3, GEODETIC ELEVATION = 559.350 m.

LEGEND	-PMEL TEST HOLE	-PMEL TEST HOLE (PIEZOMETER INSTALLED)	-PMEL TEST HOLE (FILE No. 13147)	-BENCHMARK

CONSULTING
GEOENVIRONMENTAL
GEOTECHNICAL
ENGINEERS

P. MACHIBRODA ENGINEERING LTD.

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

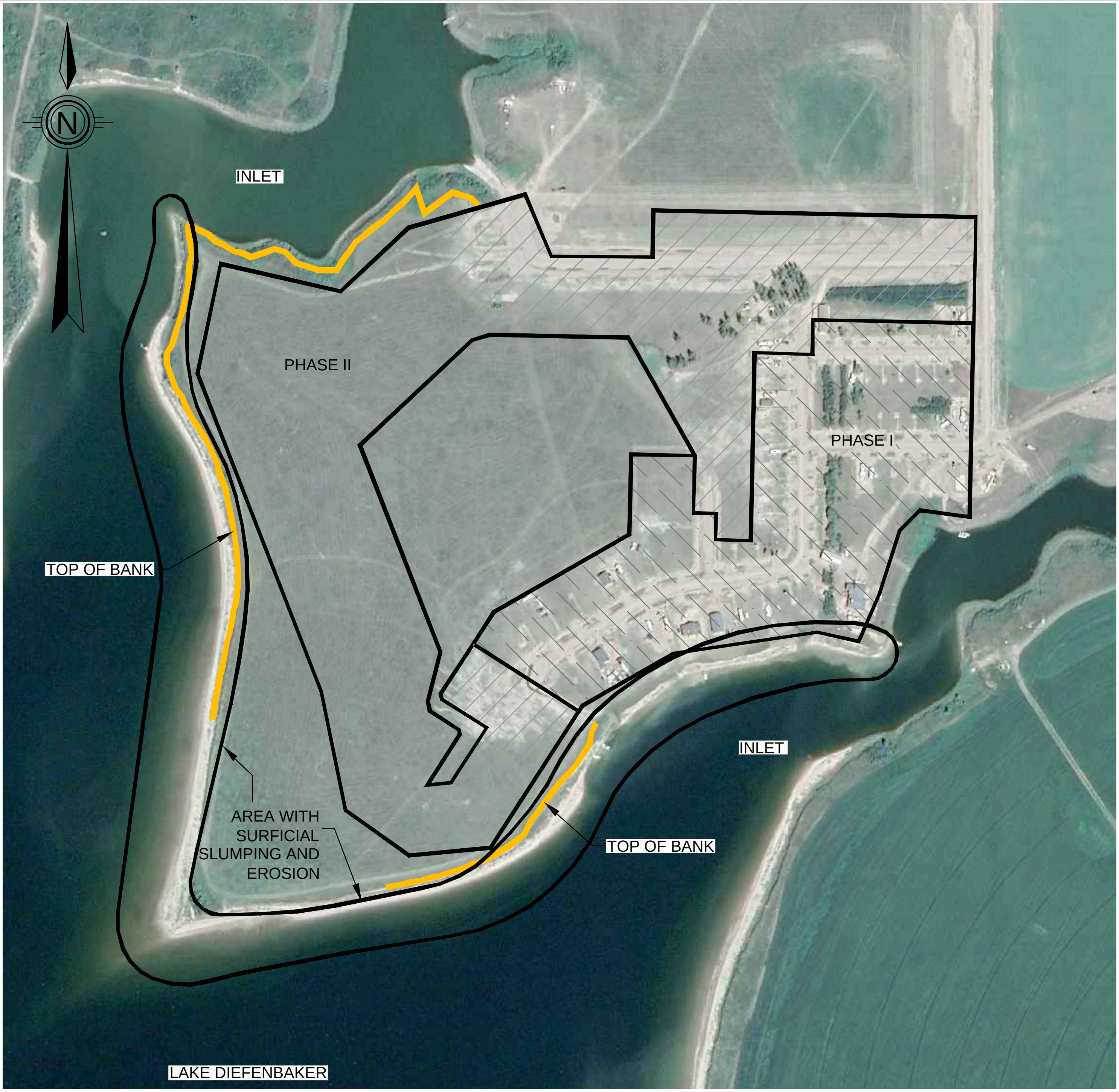
DRAWING TITLE:

SITE PLAN - TEST HOLE LOCATIONS

PROJECT:

PROPOSED SUBDIVISION - SUNSET BEACH RESORT PHASE 2
LSD 7, 9 & 10-27-25-6-W3M, RM OF LOREBURN No. 254, SK

APPROVED BY:	DRAWN BY:	DRAWING NUMBER:
GB	SD	16158-1
DATE: OCTOBER, 2019	SCALE: 1:5000	



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 ©2019, IMAGE ©2019 DIGITALGLOBE, (IMAGERY DATE: 7/26/19).

LEGEND

CONSULTING GEOENVIRONMENTAL GEOTECHNICAL ENGINEERS

P. MACHIBRODA ENGINEERING LTD.

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

DRAWING TITLE:
KEY PLAN

PROJECT:
PROPOSED SUBDIVISION - SUNSET BEACH RESORT PHASE 2
LSD 7, 9 & 10-27-25-6-W3M, RM OF LOREBURN No. 254, SK

APPROVED BY:	DRAWN BY:	DRAWING NUMBER:
GB	SD	16158-1A
DATE: OCTOBER, 2019	SCALE: 1:5000	

PIEZO. ELEV.= 561.0 m

TEST HOLE 19-1

DEPTH (m) | ELEV: 560.2 m

N	U (pp)	γ_w	Pw	Lw	w
					13.6
					10.0
	4.50				11.8
	4.00				13.3
17		21.3			14.2
	3.25		15	35	17.0
14	3.50	20.9			17.9
	3.75				18.3
23	4.00	21.0			18.0
	4.50				18.3

BENTONITE SEAL

50 mm diam. SCH 40, PVC RISER PIPE

CUTTINGS

555.7 m OCT 17/19

TOPSOIL, dark brown, moist, organics, rootlets.

SILT, some sand, trace clay, very stiff, low plastic, damp, brown.

GLACIAL TILL, clay, some silt, some sand, trace gravel, very stiff to hard, medium plastic, moist, brown.

—very stiff below 3.0 m.

—very stiff to hard below 8.0 m.

—hard below 10.0 m.

CONTINUED ON NEXT PAGE

LEGEND:

FILL, SAND, CLAY, TOPSOIL, GRAVEL, SILT, GLACIAL TILL

w.....WATER CONTENT (PERCENT OF DRY SOIL WEIGHT)

Lw...LIQUID LIMIT

Pw...PLASTIC LIMIT

γw...WET UNIT WEIGHT (kN/m^3)

U.....UNCONFINED COMPRESSIVE STRENGTH (kPa)

pp...POCKET PENETROMETER (kg/cm^2)

N.....STANDARD PENETRATION TEST (SAFETY HAMMER w/AUTOMATIC TRIP) (50/125 = BLOWS/SAMPLER PENETRATION [mm])

SO_4 .....SULPHATE CONTENT (PERCENT OF DRY SOIL WEIGHT)

P200...% PASSING No. 200 SIEVE

I.A.D.....IMMEDIATELY AFTER DRILLING

▽...RECORDED WATER LEVEL (TEST HOLE I.A.D.)

▼...RECORDED WATER LEVEL (PIEZO)

SHELBY TUBE | SPLIT SPOON | CUTTINGS

LIMITATIONS: THE FIELD DRILL LOG IS A SUMMARY OF THE SUBSURFACE CONDITIONS ENCOUNTERED AT THE SPECIFIC TEST HOLE LOCATION AT THE TIME OF TEST DRILLING. SUBSURFACE CONDITIONS MAY VARY AT OTHER LOCATIONS OF THIS SITE AND, IN TIME, MAY CHANGE AT THIS SPECIFIC TEST HOLE LOCATION.

P. MACHIBRODA ENGINEERING LTD.

FIELD DRILL LOG AND SOIL TEST RESULTS

PROJECT: PROPOSED SUBDIVISION

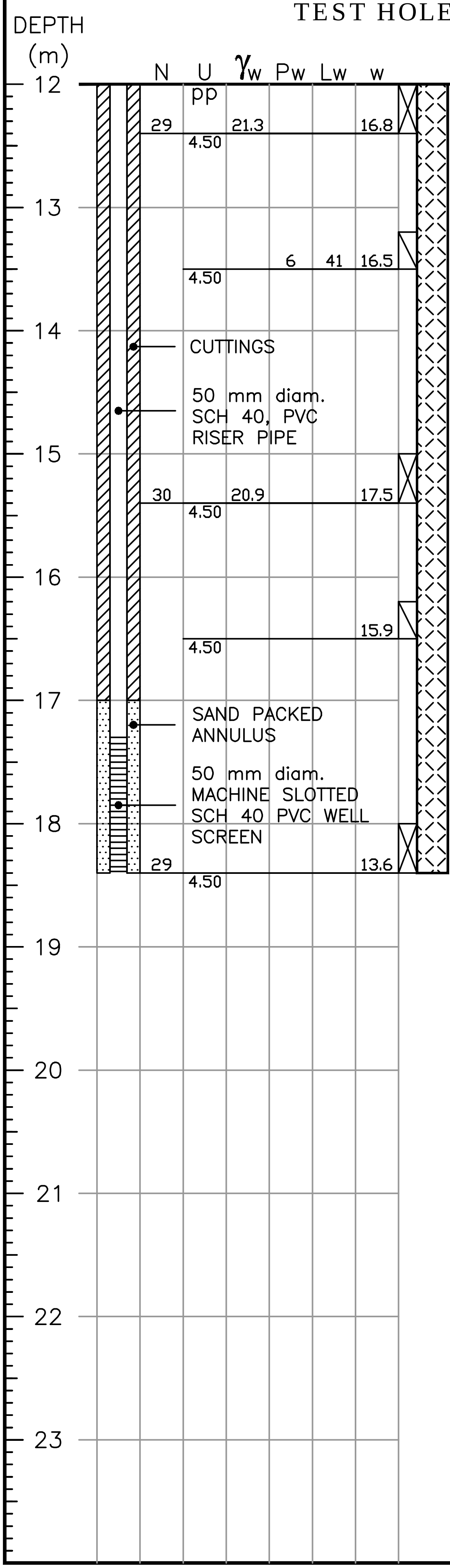
LOCATION: SUNSET BEACH RESORT PHASE 2
LSD 7, 9 & 10-27-25-6-W3M,
RM OF LOREBURN No. 254, SK

NORTHING: 5669458 EASTING: 377049

DATE DRILLED: OCT 3/19

DRAWING NUMBER: 16158-2

TEST HOLE 19-1



GLACIAL TILL, clay, some silt, some sand, trace gravel, hard, medium plastic, moist, brown.

NOTE:
1. Test Hole open and dry I.A.D.

LEGEND:

TOPSOIL, FILL, GRAVEL, SAND, SILT, CLAY, GLACIAL TILL

w.....WATER CONTENT (PERCENT OF DRY SOIL WEIGHT)

Lw...LIQUID LIMIT

Pw...PLASTIC LIMIT

γ_w...WET UNIT WEIGHT (kN/m³)

U.....UNCONFINED COMPRESSIVE STRENGTH (kPa)

pp...POCKET PENETROMETER (kg/cm²)

N.....STANDARD PENETRATION TEST (SAFETY HAMMER w/AUTOMATIC TRIP) (50/125 = BLOWS/SAMPLER PENETRATION [mm])

SO_4.....SULPHATE CONTENT (PERCENT OF DRY SOIL WEIGHT)

P200...% PASSING No. 200 SIEVE

I.A.D.....IMMEDIATELY AFTER DRILLING

▽...RECORDED WATER LEVEL (TEST HOLE I.A.D.)

▼...RECORDED WATER LEVEL (PIEZO)

SHELBY TUBE — SPLIT SPOON — CUTTINGS

LIMITATIONS: THE FIELD DRILL LOG IS A SUMMARY OF THE SUBSURFACE CONDITIONS ENCOUNTERED AT THE SPECIFIC TEST HOLE LOCATION AT THE TIME OF TEST DRILLING. SUBSURFACE CONDITIONS MAY VARY AT OTHER LOCATIONS OF THIS SITE AND, IN TIME, MAY CHANGE AT THIS SPECIFIC TEST HOLE LOCATION.

P. MACHIBRODA ENGINEERING LTD.

FIELD DRILL LOG AND SOIL TEST RESULTS

PROJECT:

PROPOSED SUBDIVISION

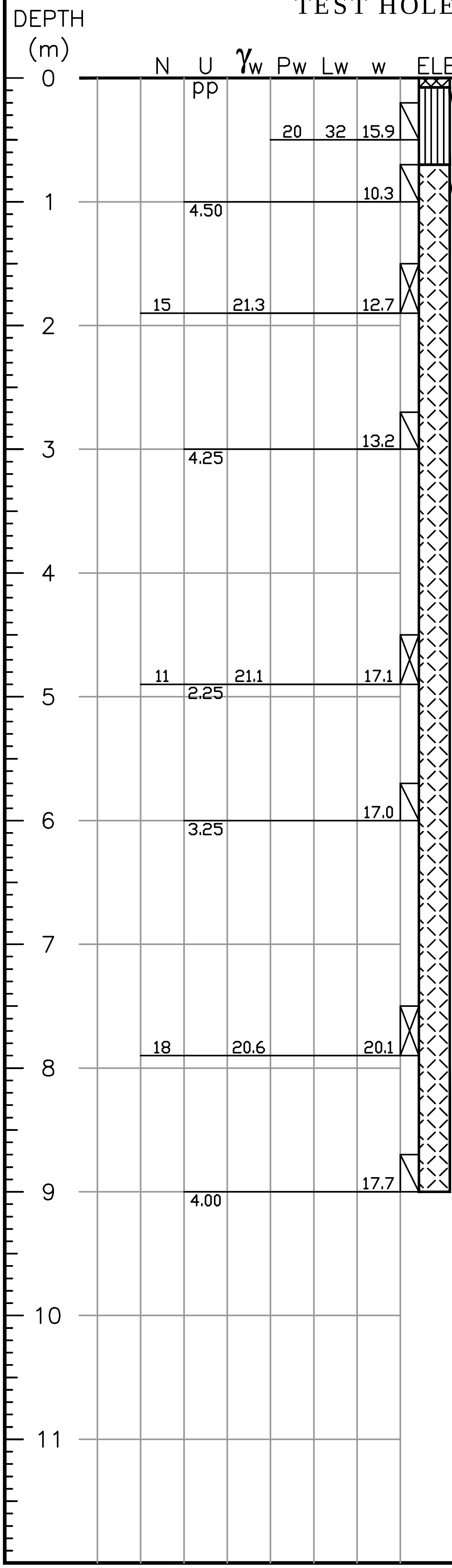
LOCATION:

SUNSET BEACH RESORT PHASE 2
LSD 7, 9 & 10-27-25-6-W3M,
RM OF LOREBURN No. 254, SK

NORTHING: 5669458 EASTING: 377049

DATE DRILLED:	DRAWING NUMBER:
OCT 3/19	16158-2A

TEST HOLE 19-2



N	U pp	γ_w	Pw	Lw	w
			20	32	15.9
	4.50				10.3
15		21.3			12.7
	4.25				13.2
11	2.25	21.1			17.1
	3.25				17.0
18		20.6			20.1
	4.00				17.7

TOPSOIL, dark brown, moist, organics, rootlets.

SILT, some sand, trace clay, hard, low to medium plastic, damp, brown.
—P200 = 74.9% at 500 mm.

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

—stiff to very stiff below 3.1 m.

—very stiff to hard below 7.5 m.

NOTE:
1. Test Hole open and dry I.A.D.

LEGEND:

FILL, SAND, CLAY, TOPSOIL, GRAVEL, SILT, GLACIAL TILL

w.....WATER CONTENT (PERCENT OF DRY SOIL WEIGHT)

Lw...LIQUID LIMIT

Pw...PLASTIC LIMIT

γ_w...WET UNIT WEIGHT (kN/m³)

U.....UNCONFINED COMPRESSIVE STRENGTH (kPa)

pp...POCKET PENETROMETER (kg/cm²)

N.....STANDARD PENETRATION TEST (SAFETY HAMMER w/AUTOMATIC TRIP) (50/125 = BLOWS/SAMPLER PENETRATION [mm])

SO_4.....SULPHATE CONTENT (PERCENT OF DRY SOIL WEIGHT)

P200...% PASSING No. 200 SIEVE

I.A.D.....IMMEDIATELY AFTER DRILLING

▽...RECORDED WATER LEVEL (TEST HOLE I.A.D.)

▼...RECORDED WATER LEVEL (PIEZO)

SHELBY TUBE   SPLIT SPOON   CUTTINGS

LIMITATIONS: THE FIELD DRILL LOG IS A SUMMARY OF THE SUBSURFACE CONDITIONS ENCOUNTERED AT THE SPECIFIC TEST HOLE LOCATION AT THE TIME OF TEST DRILLING. SUBSURFACE CONDITIONS MAY VARY AT OTHER LOCATIONS OF THIS SITE AND, IN TIME, MAY CHANGE AT THIS SPECIFIC TEST HOLE LOCATION.

P. MACHIBRODA ENGINEERING LTD.

FIELD DRILL LOG AND SOIL TEST RESULTS

PROJECT: PROPOSED SUBDIVISION

LOCATION: SUNSET BEACH RESORT PHASE 2
LSD 7, 9 & 10-27-25-6-W3M,
RM OF LOREBURN No. 254, SK

NORTHING: 5669291 EASTING: 377067

DATE DRILLED:	DRAWING NUMBER:
OCT 3/19	16158-3

PIEZO. ELEV.= 561.0 m

TEST HOLE 19-3

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

ELEV: 560.1 m

N	U / pp	γ_w	Pw	Lw	w
					11.1
					6.2
	4.50				9.8
	4.50				13.0
25		21.0			15.4
	3.25				18.3
21	2.50	20.7			16.9

BENTONITE SEAL

50 mm diam. SCH 40, PVC RISER PIPE

CUTTINGS

SAND PACKED ANNULUS

50 mm diam. MACHINE SLOTTED SCH 40 PVC WELL SCREEN

DRY OCT 17/19

TOPSOIL, dark brown, moist, organics, rootlets.

SILT, some sand, trace clay, hard, low to medium plastic, damp, brown.

GLACIAL TILL, clay, some silt, some sand, trace gravel, hard, medium plastic, damp, brown, oxide stained, gypsum crystals.
—moist below 1.2 m.

—very stiff below 3.6 m.

NOTE:
1. Test Hole open and dry I.A.D.

LEGEND:

FILL, SAND, CLAY, TOPSOIL, GRAVEL, SILT, GLACIAL TILL

w.....WATER CONTENT (PERCENT OF DRY SOIL WEIGHT)

Lw...LIQUID LIMIT

Pw...PLASTIC LIMIT

γ_w...WET UNIT WEIGHT (kN/m³)

U.....UNCONFINED COMPRESSIVE STRENGTH (kPa)

pp...POCKET PENETROMETER (kg/cm²)

N.....STANDARD PENETRATION TEST (SAFETY HAMMER w/AUTOMATIC TRIP) (50/125 = BLOWS/SAMPLER PENETRATION [mm])

SO_4.....SULPHATE CONTENT (PERCENT OF DRY SOIL WEIGHT)

P200...% PASSING No. 200 SIEVE

I.A.D.....IMMEDIATELY AFTER DRILLING

▽...RECORDED WATER LEVEL (TEST HOLE I.A.D.)

▼...RECORDED WATER LEVEL (PIEZO)

SHELBY TUBE, SPLIT SPOON, CUTTINGS

LIMITATIONS: THE FIELD DRILL LOG IS A SUMMARY OF THE SUBSURFACE CONDITIONS ENCOUNTERED AT THE SPECIFIC TEST HOLE LOCATION AT THE TIME OF TEST DRILLING. SUBSURFACE CONDITIONS MAY VARY AT OTHER LOCATIONS OF THIS SITE AND, IN TIME, MAY CHANGE AT THIS SPECIFIC TEST HOLE LOCATION.

P. MACHIBRODA ENGINEERING LTD.

FIELD DRILL LOG AND SOIL TEST RESULTS

PROJECT: PROPOSED SUBDIVISION

LOCATION: SUNSET BEACH RESORT PHASE 2 LSD 7, 9 & 10-27-25-6-W3M, RM OF LOREBURN No. 254, SK

NORTHING: 5669141 EASTING: 377205

DATE DRILLED: OCT 3/19

DRAWING NUMBER: 16158-4

TEST HOLE 19-4

DEPTH (m): 0 – 11

ELEV: 559.5 m

N	U (pp)	γ_w	Pw	Lw	w
			22	33	17.4
					12.6
14		20.1			13.9
	4.00		21	35	14.8
16	2.50	21.1			16.4
	3.25				17.1
22	3.25	20.7			18.2
	4.00				19.4

TOPSOIL, dark brown, moist, organics, rootlets.

SILT, some sand, trace clay, very stiff to hard, low to medium plastic, moist, brown.
–P200 = 88.7% at 500 mm.

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

–very stiff to hard below 2.3 m.

–very stiff below 3.6 m.

–very stiff to hard below 8.4 m.

NOTE:
1. Test Hole open and dry I.A.D.

LEGEND:

FILL, SAND, CLAY, TOPSOIL, GRAVEL, SILT, GLACIAL TILL

w.....WATER CONTENT (PERCENT OF DRY SOIL WEIGHT)

Lw...LIQUID LIMIT

Pw...PLASTIC LIMIT

γ_w...WET UNIT WEIGHT (kN/m³)

U.....UNCONFINED COMPRESSIVE STRENGTH (kPa)

pp...POCKET PENETROMETER (kg/cm^2)

N.....STANDARD PENETRATION TEST (SAFETY HAMMER w/AUTOMATIC TRIP) (50/125 = BLOWS/SAMPLER PENETRATION [mm])

SO_4.....SULPHATE CONTENT (PERCENT OF DRY SOIL WEIGHT)

P200...% PASSING No. 200 SIEVE

I.A.D.....IMMEDIATELY AFTER DRILLING

▽...RECORDED WATER LEVEL (TEST HOLE I.A.D.)

▼...RECORDED WATER LEVEL (PIEZO)

SHELBY TUBE   SPLIT SPOON   CUTTINGS

LIMITATIONS: THE FIELD DRILL LOG IS A SUMMARY OF THE SUBSURFACE CONDITIONS ENCOUNTERED AT THE SPECIFIC TEST HOLE LOCATION AT THE TIME OF TEST DRILLING. SUBSURFACE CONDITIONS MAY VARY AT OTHER LOCATIONS OF THIS SITE AND, IN TIME, MAY CHANGE AT THIS SPECIFIC TEST HOLE LOCATION.

P. MACHIBRODA ENGINEERING LTD.

FIELD DRILL LOG AND SOIL TEST RESULTS

PROJECT: PROPOSED SUBDIVISION

LOCATION: SUNSET BEACH RESORT PHASE 2
LSD 7, 9 & 10-27-25-6-W3M,
RM OF LOREBURN No. 254, SK

NORTHING: 5668995 EASTING: 377308

DATE DRILLED: OCT 3/19

DRAWING NUMBER: 16158-5

TEST HOLE 19-5

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

ELEV: 561.6 m

N	U / pp	γ_w	Pw	Lw	w
					10.5
					9.6
					8.2
					9.4
29		21.6			12.1
					15.0
22		21.4			16.3

TOPSOIL, dark brown, moist, organics, rootlets.

SILT, some sand, some clay, very stiff, low to medium plastic, damp, brown.

GLACIAL TILL, clay, some silt, some sand, trace gravel, hard, medium plastic, damp, brown, oxide stained, gypsum crystals.

–moist below 2.5 m.

–very stiff below 3.8 m.

NOTE:
1. Test Hole open and dry I.A.D.

LEGEND:

FILL, SAND, CLAY, TOPSOIL, GRAVEL, SILT, GLACIAL TILL

w.....WATER CONTENT (PERCENT OF DRY SOIL WEIGHT)

Lw...LIQUID LIMIT

Pw...PLASTIC LIMIT

γ_w...WET UNIT WEIGHT (kN/m^3)

U.....UNCONFINED COMPRESSIVE STRENGTH (kPa)

pp...POCKET PENETROMETER (kg/cm^2)

N.....STANDARD PENETRATION TEST (SAFETY HAMMER w/AUTOMATIC TRIP) (50/125 = BLOWS/SAMPLER PENETRATION [mm])

SO_4 .....SULPHATE CONTENT (PERCENT OF DRY SOIL WEIGHT)

P200...% PASSING No. 200 SIEVE

I.A.D.....IMMEDIATELY AFTER DRILLING

▽...RECORDED WATER LEVEL (TEST HOLE I.A.D.)

▼...RECORDED WATER LEVEL (PIEZO)

SHELBY TUBE   SPLIT SPOON   CUTTINGS

LIMITATIONS: THE FIELD DRILL LOG IS A SUMMARY OF THE SUBSURFACE CONDITIONS ENCOUNTERED AT THE SPECIFIC TEST HOLE LOCATION AT THE TIME OF TEST DRILLING. SUBSURFACE CONDITIONS MAY VARY AT OTHER LOCATIONS OF THIS SITE AND, IN TIME, MAY CHANGE AT THIS SPECIFIC TEST HOLE LOCATION.

P. MACHIBRODA ENGINEERING LTD.

FIELD DRILL LOG AND SOIL TEST RESULTS

PROJECT: PROPOSED SUBDIVISION

LOCATION: SUNSET BEACH RESORT PHASE 2 LSD 7, 9 & 10-27-25-6-W3M, RM OF LOREBURN No. 254, SK

NORTHING: 5669383 EASTING: 377252

DATE DRILLED: OCT 3/19

DRAWING NUMBER: 16158-6

PIEZO. ELEV.= 563.9 m

TEST HOLE 19-6

DEPTH (m)

ELEV: 562.9 m

N	U (pp)	γw	Pw	Lw	w
			21	35	11.9
					11.1
					14.9
	2.75				15.4
20		21.1			14.2
	3.25				13.6
24		21.0			16.1
	3.75				15.6
27		21.5			15.6

BENTONITE SEAL

50 mm diam. SCH 40, PVC RISER PIPE

CUTTINGS

SAND PACKED ANNULUS

50 mm diam. MACHINE SLOTTED SCH 40 PVC WELL SCREEN

554.0 m
OCT 17/19

0 1 2 3 4 5 6 7 8 9 10 11

SILT, some sand, trace clay, very stiff, low to medium plastic, moist, dark brown.
–damp, brown below 500 mm.

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

–very stiff stiff to hard below 7.8 m.

NOTE:
1. Test Hole open and dry I.A.D.

LEGEND:

TOPSOIL FILL GRAVEL SAND SILT CLAY GLACIAL TILL

w.....WATER CONTENT (PERCENT OF DRY SOIL WEIGHT)

Lw...LIQUID LIMIT

Pw...PLASTIC LIMIT

γw...WET UNIT WEIGHT (kN/m³)

U.....UNCONFINED COMPRESSIVE STRENGTH (kPa)

pp...POCKET PENETROMETER (kg/cm²)

N.....STANDARD PENETRATION TEST (SAFETY HAMMER w/AUTOMATIC TRIP) (50/125 = BLOWS/SAMPLER PENETRATION [mm])

SO_4 .....SULPHATE CONTENT (PERCENT OF DRY SOIL WEIGHT)

P200...% PASSING No. 200 SIEVE

I.A.D.....IMMEDIATELY AFTER DRILLING

▽...RECORDED WATER LEVEL (TEST HOLE I.A.D.)

▼...RECORDED WATER LEVEL (PIEZO)

SHELBY TUBE SPLIT SPOON CUTTINGS

LIMITATIONS: THE FIELD DRILL LOG IS A SUMMARY OF THE SUBSURFACE CONDITIONS ENCOUNTERED AT THE SPECIFIC TEST HOLE LOCATION AT THE TIME OF TEST DRILLING. SUBSURFACE CONDITIONS MAY VARY AT OTHER LOCATIONS OF THIS SITE AND, IN TIME, MAY CHANGE AT THIS SPECIFIC TEST HOLE LOCATION.

P. MACHIBRODA ENGINEERING LTD.

FIELD DRILL LOG AND SOIL TEST RESULTS

PROJECT:
PROPOSED SUBDIVISION

LOCATION:
SUNSET BEACH RESORT PHASE 2
LSD 7, 9 & 10-27-25-6-W3M,
RM OF LOREBURN No. 254, SK

NORTHING: 5669461 EASTING: 377541

DATE DRILLED:	DRAWING NUMBER:
OCT 3/19	16158-7

APPENDIX A

Explanation of Terms on
Test Hole 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.

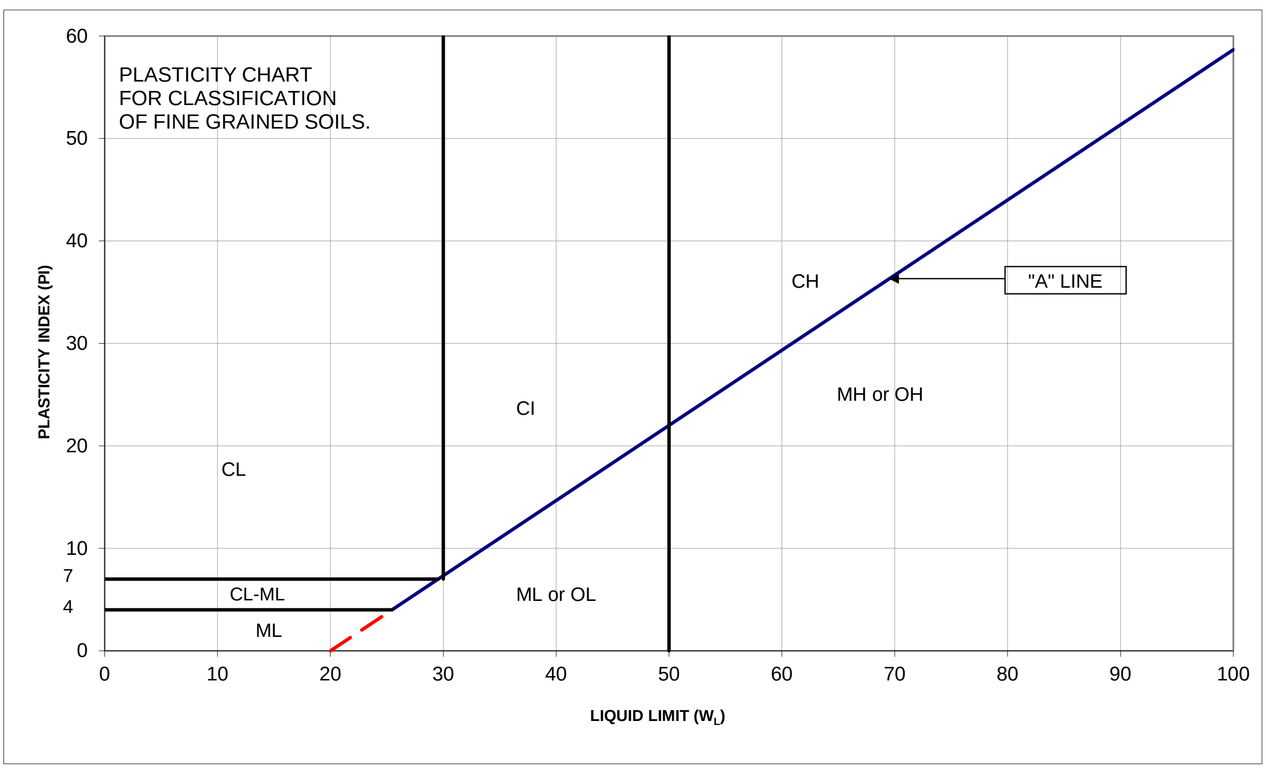
Organic Soils: Readily identified by colour, odour, spongy feel and frequently by fibrous texture.

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	$W_L > 50$



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APPENDIX B

Grain Size Distribution
Analysis Results

P.MACHIBRODA ENGINEERING LTD.

ASTM D422: PARTICLE SIZE ANALYSIS OF SOILS

Project: Proosed Subdivision - Sunset Beach Resort Phase 2

Location: LSD 7, 9 & 10-2725-6W3M, RM of Loreburn No. 254, SK

Project No.: 16158

Date Tested: October 15, 2019

Test Hole No.: 19-1

Sample No.: 6

Depth (m): 4.5

Sieve Analysis:

Sieve	Diameter mm	% Finer
1.5"	38.1	100
1"	25.4	100
3/4"	19.1	100
1/2"	12.7	100
3/8"	9.5	99
# 4	4.75	99
# 10	2	98
# 20	0.85	95
# 40	0.425	90.2
#60	0.25	84.1
# 100	0.15	76.3
# 200	0.075	64.1

Hydrometer Analysis:

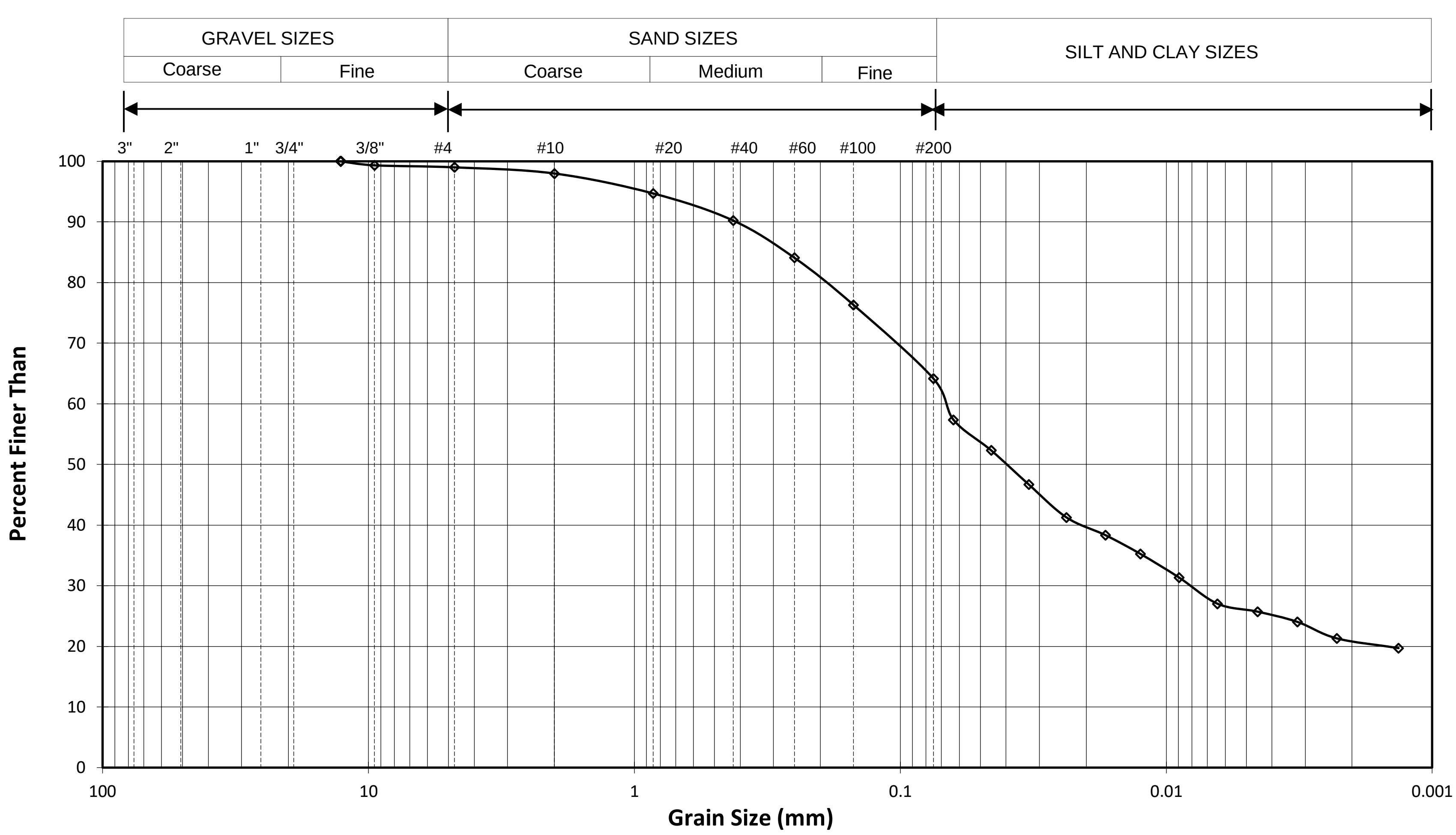
Dispersing Agent:
Sodium Hexametaphosphate

Diameter mm	% Finer
0.0631	57.4
0.0455	52.3
0.0329	46.7
0.0237	41.3
0.0169	38.3
0.0125	35.2
0.0090	31.4
0.0064	27.0
0.0045	25.7
0.0032	24.0
0.0023	21.3
0.0013	19.7

Material Description:

% Gravel Sizes	% Sand Sizes	% Silt Sizes	% Clay Sizes
1	35	43	21

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 ASTM D422 STANDARD
P. MACHIBRODA ENGINEERING LTD.

PER Preston Scherguitch

P.MACHIBRODA ENGINEERING LTD.

ASTM D422: PARTICLE SIZE ANALYSIS OF SOILS

Project: Proosed Subdivision - Sunset Beach Resort Phase 2
Location: LSD 7, 9 & 10-2725-6W3M, RM of Loreburn No. 254, SK
Project No.: 16158
Date Tested: October 15, 2019
Test Hole No.: 19-1
Sample No.: 12
Depth (m): 13.5

Sieve Analysis:

Sieve	Diameter mm	% Finer
1.5"	38.1	100
1"	25.4	100
3/4"	19.1	100
1/2"	12.7	100
3/8"	9.5	99
# 4	4.75	98
# 10	2	96
# 20	0.85	94
# 40	0.425	89.1
#60	0.25	82.4
# 100	0.15	74.6
# 200	0.075	63.8

Hydrometer Analysis:

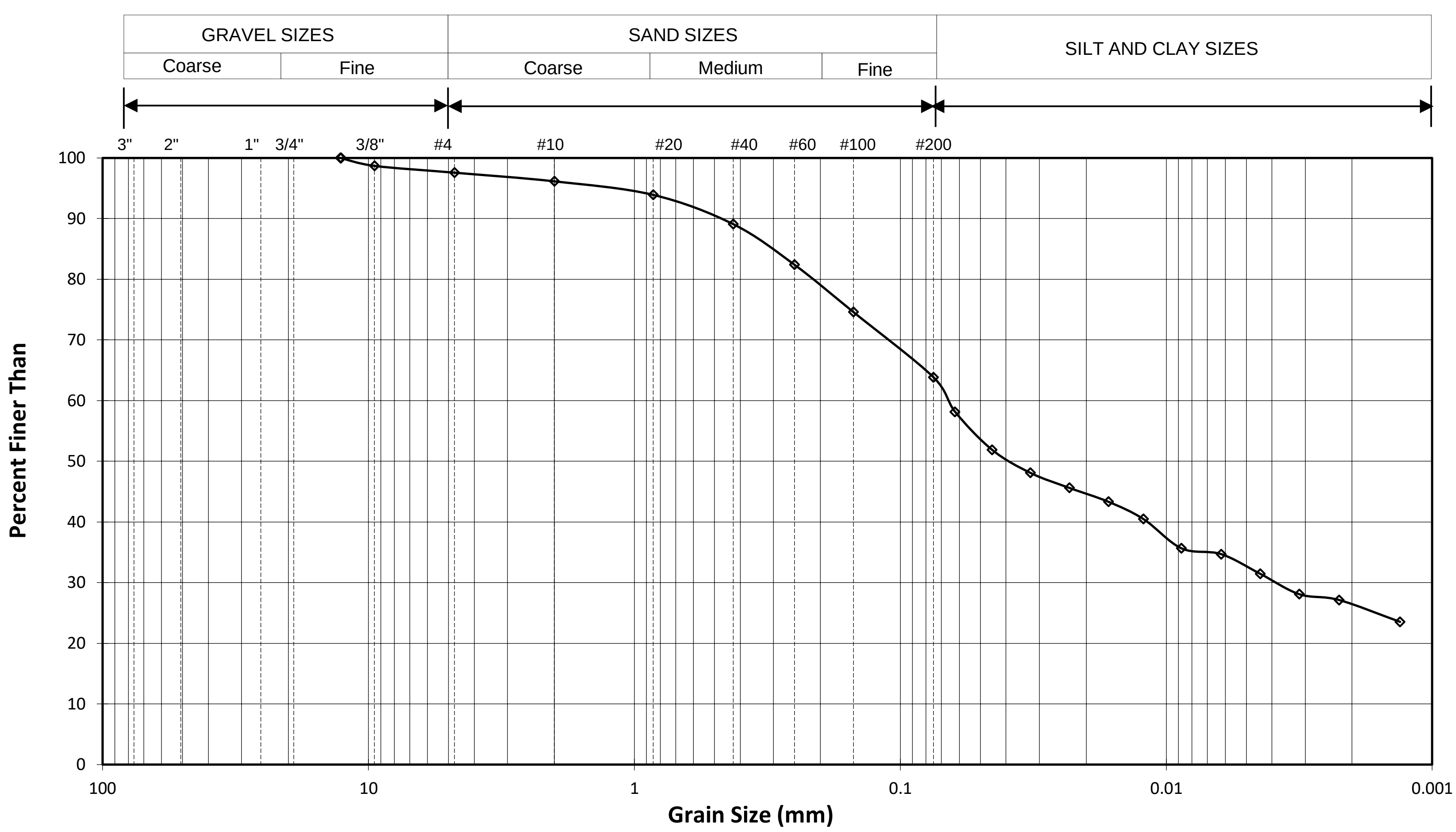
Dispersing Agent:
Sodium Hexametaphosphate

Diameter mm	% Finer
0.0623	58.2
0.0452	51.9
0.0324	48.1
0.0231	45.6
0.0165	43.3
0.0122	40.5
0.0088	35.6
0.0062	34.7
0.0044	31.5
0.0032	28.1
0.0022	27.1
0.0013	23.5

Material Description:

% Gravel Sizes	% Sand Sizes	% Silt Sizes	% Clay Sizes
2	34	37	27

Remarks:



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Drawing No.

Appendix B-2

WE CERTIFY TESTING PROCEDURES ARE IN ACCORDANCE WITH ASTM D422 STANDARD
P. MACHIBRODA ENGINEERING LTD.
PER Preston Scherqevitch

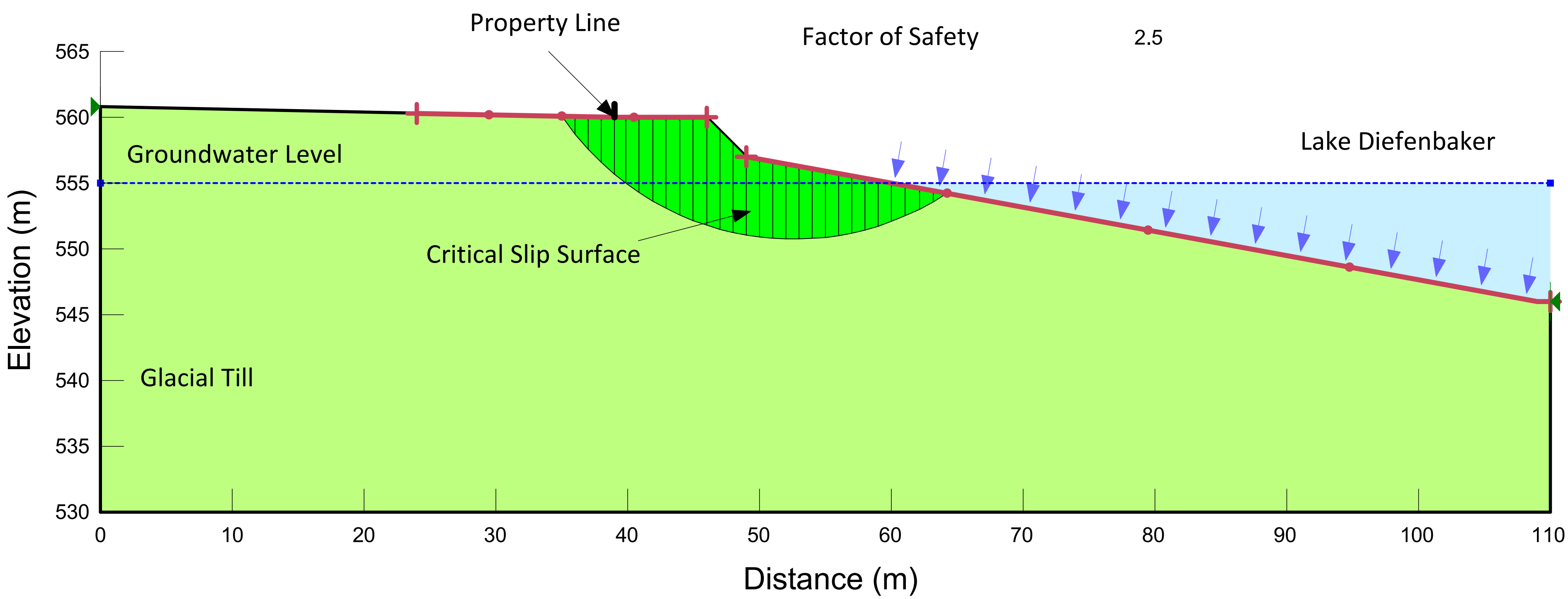
APPENDIX C

Slope Stability Plots

PMEL File No. 16158

Drawing No. Appendix C-1

SECTION AA-AA'



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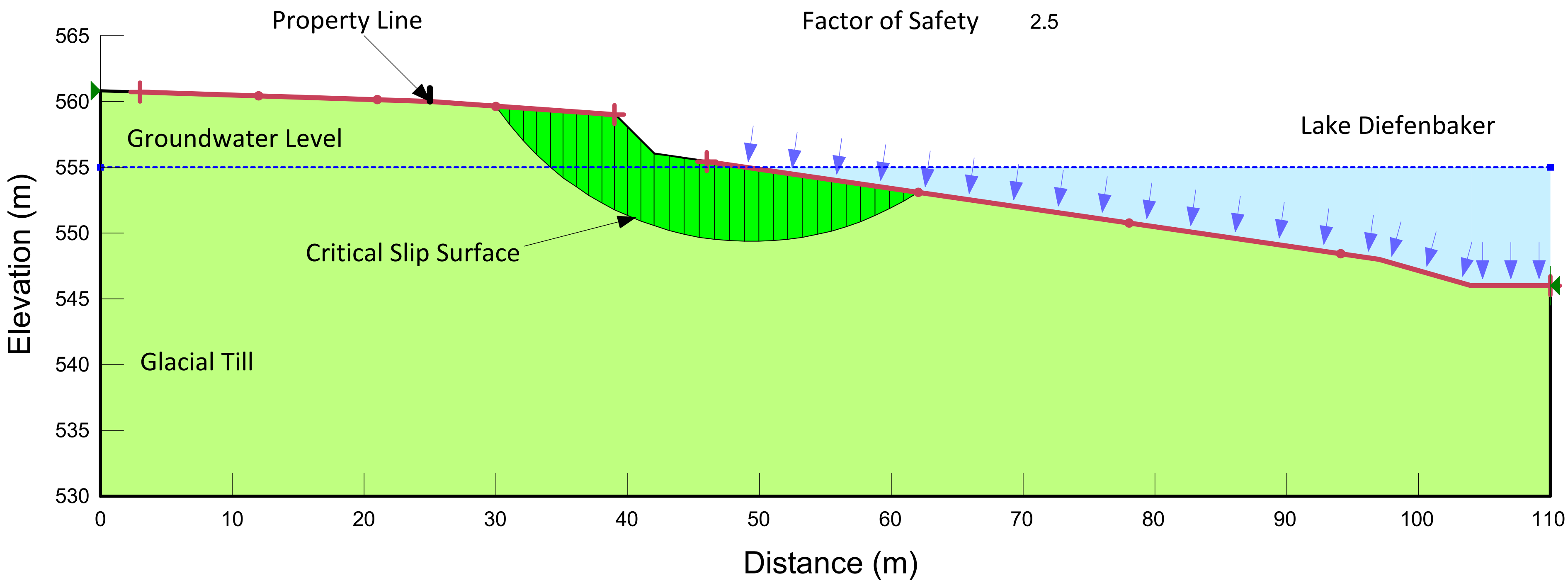
SECTION BB-BB'



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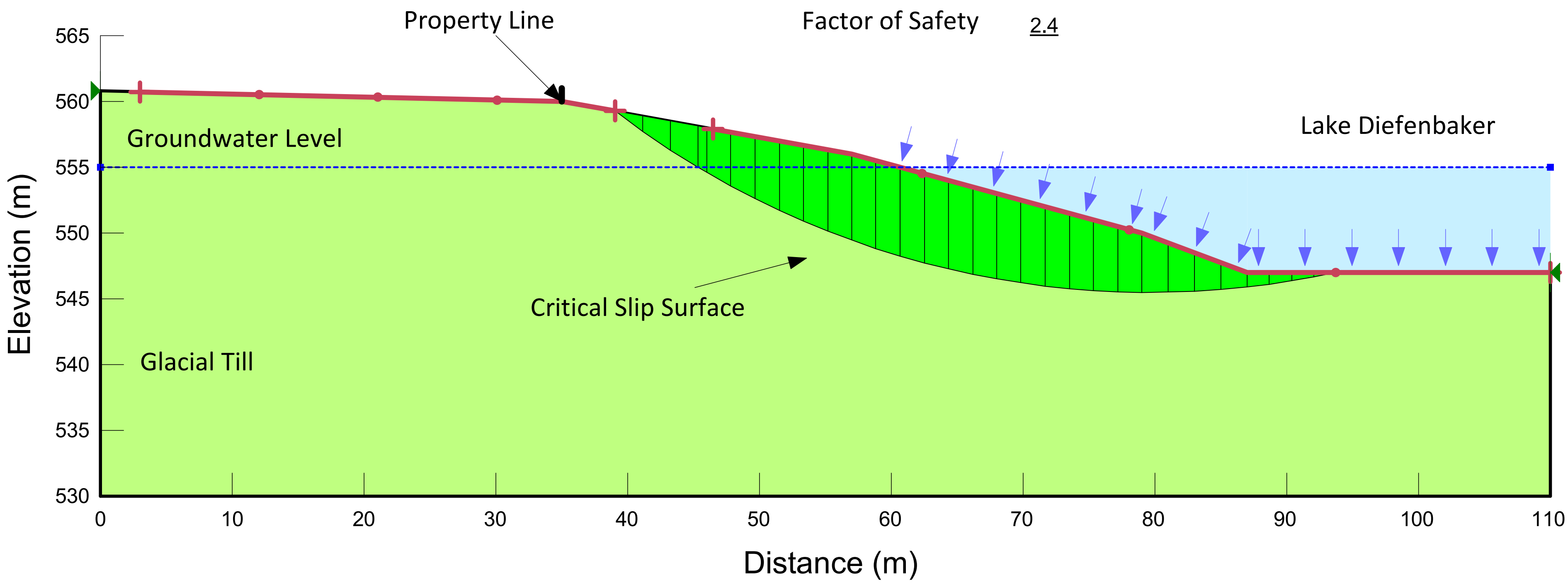
PMEL File No. 16158

Drawing No. Appendix C-3

SECTION CC-CC'



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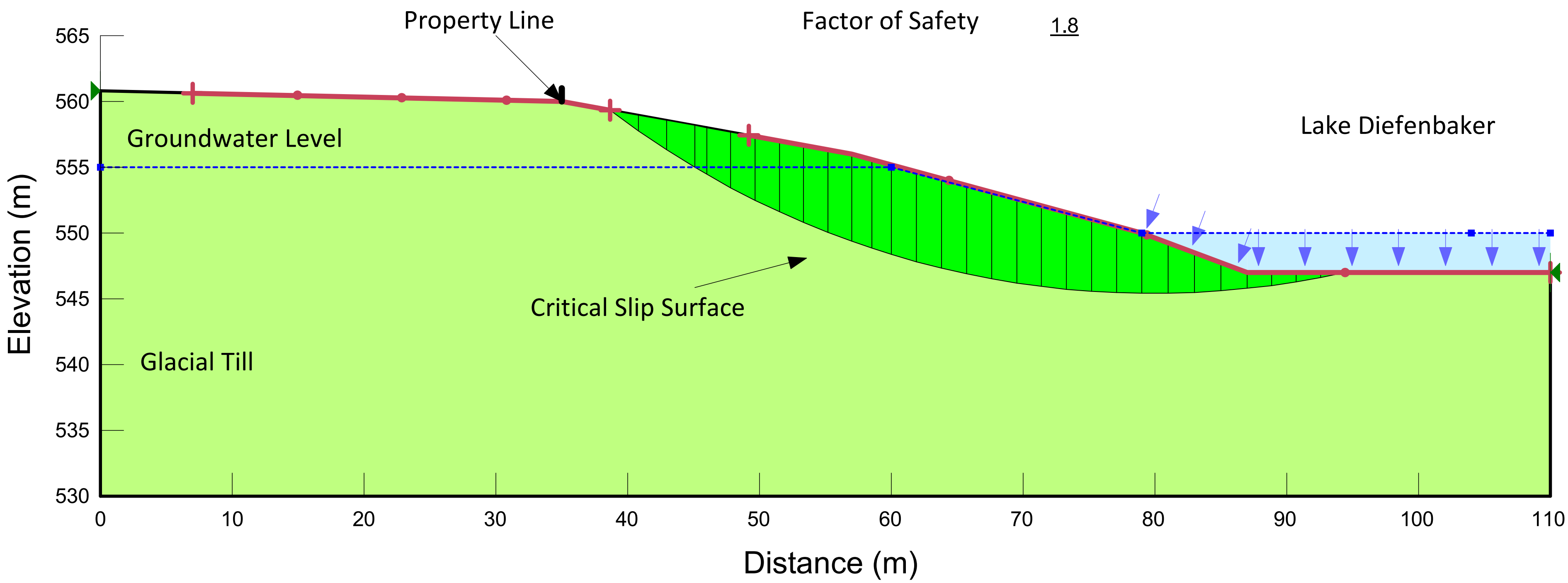
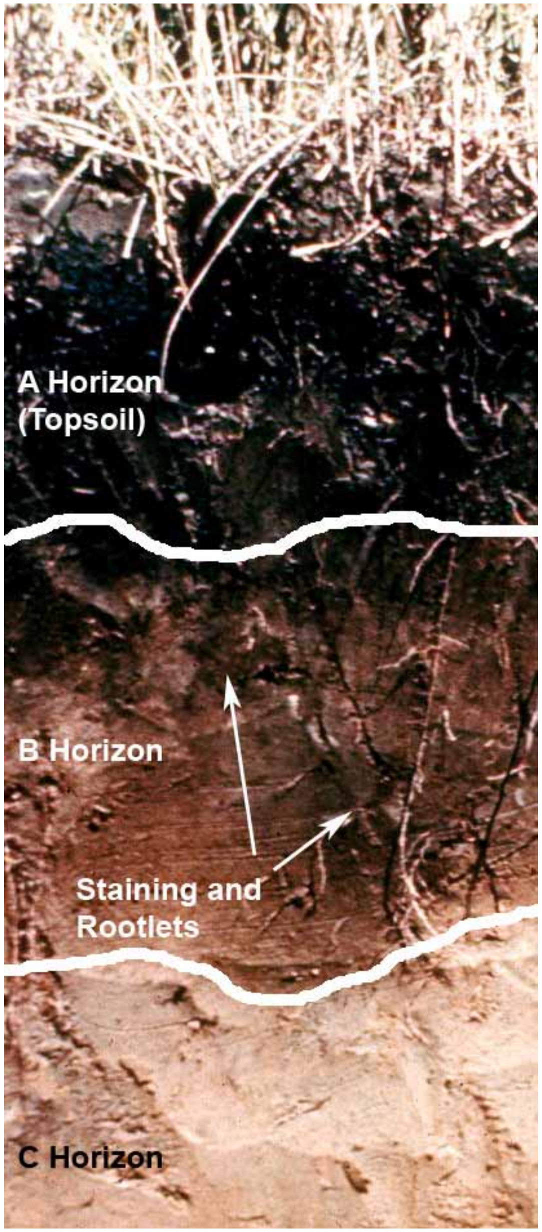
SECTION CC-CC'
5 m Drop in Lake Level



APPENDIX D

Topsoil, Organics
and Organic Matter



A Horizon

The A horizon is the topsoil layer of the soil strata. It is characterized by a build up of organic matter, and a lower unit weight than subsequent layers. The organic matter content of this layer is typically 4-10% by mass.

The colour of this horizon varies from dark black to brown, depending on surface vegetation and climatic conditions.

B Horizon

Typically reddish brown in colour and contains accumulations of matter that have been washed down from the A Horizon. The B horizon is generally composed of clay that has been washed out of the A Horizon, but can also contain iron, calcium and sodium deposits as well.

C Horizon

Unweathered parent soil.

Topsoil is a mixture of mineral soil and organic matter. The organic matter is developed from decaying biological material (leaves, grass, trees, animals, etc.) and contributes to the brown to black colour of the soil. Following the topsoil is the B horizon which is a transition layer, where staining from the overlying topsoil is common. This results in a darker colour of the soil immediately below the organic topsoil layer. Depending on the surface vegetation, rootlets may be present below the depth of topsoil. However it should be recognized that these rootlets are not the same as organic matter in topsoil.

Physically speaking in comparison to mineral soil, topsoil has a significantly lower bulk density and a lower unit weight as compared to the underlying parent soil. This is due to larger pore spaces and non mineral materials in the soil matrix. Along with lower density, topsoil is often spongy and colloidal/fibrous. The following figure is of a typical prairie soil. Each horizon is labelled accordingly to demonstrate a typical soil profile.

Reference

Henry L. 2003. Henry's Handbook of Soil and Water, Henry Perspectives, Saskatoon, SK.