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GEOTECHNICAL INVESTIGATION AND SLOPE STABILITY ASSESSMENT PROPOSED SUBDIVISION - PHASE 1 PART OF 27-25-6-W3M RM OF LOREBURN NO. 254, SASKATCHEWAN

PMEL FILE NO. 13147

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1.0 INTRODUCTION

The following report has been prepared on the results of a geotechnical investigation and slope stability analysis conducted for Phase I of the proposed new Subdivision located within 27-25-6-W3M, in the RM of Loreburn No. 254, Saskatchewan. Phase I of the proposed Subdivision is located within parts of L.S. 8 and L.S. 9 of 27-25-6-W3M.

Authorization to proceed with this investigation was provided on September 18, 2017, via the signed P. Machibroda Engineering Ltd. (PMEL) Consulting Agreement. The Terms of Reference for this investigation were presented in PMEL Proposal No. 13147REV1, dated September 18, 2017.

The field test drilling and soil sampling were conducted on October 10, 2017. A visual review of the subject site was conducted on September 28, 2017. Groundwater monitoring was completed on October 30, 2017.

2.0 FIELD INVESTIGATION

Five (5) test holes, located as shown on the Site Plan, Drawing No. 13147-1, were dry drilled using our truck mounted, continuous flight, solid stem auger drill rig. The test holes were 150 mm in diameter and were extended to depths of 9 to 15.4 metres below the existing ground surface.

Test hole drill logs 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) were installed in the Test Holes to monitor the groundwater conditions.

3.0 FIELD DRILL LOGS

The field drill logs recorded during test drilling have been shown plotted on Drawing Nos. 13147-2 to 6A, inclusive.

The ground surface elevation at each Test Hole location was referenced to the top of a found iron pin (legal survey marker), located within the northeast corner of SE-27-25-6-W3M approximately as shown on the Site Plan, Drawing No. 13147-1. An orthometric elevation of 562.687 m was provided by Associated Engineering for the top of pin

3.1 Soil Profile

The general soil profile consisted of a surficial layer of topsoil (approximately 100 mm thick) overlying silt followed by glacial till which extended to a depth of at least 15.4 metres below existing grade, the maximum depth drilled with our testing at this site. Sand was encountered at the bottom of Test Hole No. 15-1 underlying the glacial till.

3.2 Groundwater Conditions, Sloughing

The Test Holes remained open during and immediately following test drilling.

The depths at which groundwater seepage was encountered has been shown on Drawing Nos. 13147-2 to 6A, inclusive. A summary of the measured groundwater elevations recorded in the installed piezometers is presented in Table I.

TABLE I. RECORDED GROUNDWATER LEVELS

Test Hole No.	Piezometer Rim Elevation (metres)	Ground Surface Elevation (metres)	Groundwater Elevation (metres)	
			Immediately after Installation	October 30, 2017
17-1	562.9	561.9	Trace	555.7
17-2	566.2	565.2	Dry	560.9
17-3	563.6	562.5	Trace	554.5
17-4	562.6	561.5	Dry	553.3
17-5	561.4	560.4	Dry	545.4

An examination of Table I revealed the groundwater level was situated between approximately 4.3 and 15 metres below existing grade on October 30, 2017. The piezometer may not have stabilized and higher or perched groundwater conditions should be anticipated during and/or following precipitation events and/or snow melt.

4.0 LABORATORY ANALYSIS

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

The results of soil classification and index tests conducted on representative samples of soil recovered from this site have been plotted alongside the depth at which the samples were recovered as shown on Drawing Nos. 13147-2 to 6A, inclusive.

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

5.0 SLOPE STABILITY

The theoretical slope stability analysis was performed using the SLOPE/W computer program available through Geo-Slope International Ltd.[1]

[1] Geo-Slope International Ltd., 2007. Slope/W User's Manual, A Comprehensive Program for Slope Stability Analysis, Geo-Slope International Ltd., Calgary, Alberta.

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 Site/Project Description

PMEL conducted a visual site review of the site on September 28, 2017. Select photographs taken during the site review have been presented in Appendix D. An area plan of the subject site has been shown on Drawing No. 13147-1A.

The subject site was vacant pasture land, covered with grasses with some trees (shelter belt from an old farmyard) in the northeast corner of Phase I of the proposed Subdivision. The subject site is bordered by inlets of Lake Diefenbaker to the south/southeast and northeast, Lake Diefenbaker to the west and agricultural land (cultivated and pasture) to the north. Phase I of the proposed Subdivision is located along a north shoreline of an inlet. The proposed Boat Launch is located on a south shoreline of another inlet located north of the subject site.

In general, 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.

5.2 Aerial Photograph Review

Historical aerial photographs and satellite imagery of the subject site from 1965, 1988, 1996, 2007 and 2013 were reviewed. Based on the aerial photographs the subject site appears to have remained relatively unchanged since at least 1965. Based on the 1965 aerial photographs, historical landslides were evident on the old valley slopes (now underwater). However, above the original crest of slope (present day shoreline) there was no apparent evidence of slope movement prior to 1965 and between 1965 to present day. Some erosion was evident along the shoreline since at least 1988. The magnitude of erosion could be not determined but did not appear to have significantly altered the shoreline between 1988 and present day.

5.3 Input for Analysis

5.3.1 Surface Geometry

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 A-A', B-B' and C-C', located approximately as shown on Drawing No. 13147-1.

5.3.2 Soil Stratigraphy

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

5.3.3 Piezometric Conditions

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 the 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 metres through the year (has varied approximately 3.5 metres over 2017)[2].

5.3.4 Soil Properties

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 INPUT 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

5.4 Results of Analysis

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

TABLE III. SLOPE STABILITY ANALYSES RESULTS

Slope Section	Piezometric Condition	Calculated Factor of Safety	Drawing No.
A-A’	Inferred	2.9	C-1
B-B’	Inferred	2.8	C-2
C-C’	Inferred	2.9	C-3
B-B’	5 metre drop in lake water level	2.0	C-4

[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 November 3, 2017.

Based on the slope stability analyses the Factor of Safety at the back property line (lakeside) of the proposed lots was at least 2.9 and 2.8 for Slope Sections A-A' and B-B', respectively. At the proposed boat launch location (Slope Section C-C') the Factor of Safety of the existing slope was 2.9.

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 metre drop in water level the Factor of Safety at Slope Section B-B' dropped from 2.8 to 2.0.

5.5 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 slope was at least 2.8, 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 30 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.

However, surficial slumping and erosion along the shoreline was evident during PMEL visual site review, particularly in the area of proposed Block 1, Lot Nos. 7 to 16, inclusive (refer to Drawing No. 13147-1A). 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.

The Factor of Safety at the proposed Boat Launch location (Slope Section C-C') was 2.9. As such, the slope at this location was considered stable. It is recommended that the proposed Boat Launch grading plan is reviewed by PMEL once completed.

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 double walled holding tank and disposed off-site. The tank should be located well away from the crest of the slope.

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.

6.0 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 general soil profile consisted of organic topsoil overlying silt followed by glacial till. The groundwater level was measured between 4.3 and 15 metres below existing ground surface on October 30, 2017. Higher groundwater levels should be expected during periods of precipitation and during spring thaw.

The subgrade soils are frost susceptible and the average depth of frost penetration for a heated building in the RM of Loreburn, Saskatchewan area is approximately 1.5 metres.

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 and dewatering; standard strip and/or spread footings; drilled, cast-in-place concrete piles; helical, screw piles; floor slabs; foundation walls; grade beams; foundation concrete and roadway structures.

6.2 Site Preparation

All organic topsoil and deleterious materials should be removed from the building footprint. 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.

See Appendix E for further information with respect to topsoil composition and soil structure.

The surface of the subgrade should be levelled and compacted to the following minimum density requirements.

Building Areas	-	96 percent of standard Proctor density at optimum moisture content;
Roadway Areas	-	96 percent of standard Proctor density at optimum moisture content;
Landscape Areas	-	90 percent of standard Proctor density at optimum moisture content.

Subgrade fill, if required, should preferably consist of granular material or non-expansive (i.e., low plastic) fine grained soils. The on-site glacial till soils are considered suitable for use as subgrade fill. The fill should be placed in thin lifts (maximum 150 mm loose) and compacted to 96 percent of standard Proctor density at optimum moisture content. All proposed subgrade fill should be approved by the Geotechnical Consultant prior to placement.

The site should be graded to ensure positive site drainage away from all structures and away from the crest of slope.

6.3 Excavations and Dewatering

Excavations at this site should be made in accordance with current Saskatchewan Labour Occupational Health and Safety (OH&S) Guidelines. The subsurface conditions at this site may be classified as “Type 3” soils. Sideslopes should be sloped at a minimum of 1H:1V (Horizontal:Vertical). Slopes should be flattened where groundwater seepage or sloughing conditions are encountered.

Depending on lateral constraints, excavations at this site may be completed with unbraced, sloped side walls. The stability of the excavation walls will be affected by wetting and drying of the exposed excavation walls, the length of time that the excavation remains open and the consistency and structure (degree of fracturing, slickensiding, etc.) of the subgrade soils. The excavated soil should be removed from the excavation banks (and stockpiled) to minimize potential sloughing of the trench sidewalls due to the soil surcharge loading.

The groundwater level was measured between 4.3 and 15 metres below existing ground surface on October 30, 2017 which is well below the anticipated depth of a basement (i.e., 1.5 metres below existing grade). As such, groundwater seepage is not anticipated to impact shallow excavations at this site. De-watering of the excavations should be conducted on an “as required” basis where groundwater seepage is encountered or precipitation enters the excavation.

6.4 Standard Strip and/or Spread Footings

A footing foundation based on naturally deposited, undisturbed soil 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.

1. Footings should be founded on naturally deposited, undisturbed glacial till below the average depth of frost penetration (i.e., approximately 1.5 metres below finished ground surface for a year round heated structure and 2.0 metres for an unheated structure). The footing excavations should be hand-cleaned to remove all loose, disturbed soil, and, to expose naturally deposited, undisturbed soil.

2. If insulation is utilized beneath the basement floor slab, it is recommended to leave a one (1) metre 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 area is insulated.

3. Footings, based on naturally deposited, undisturbed glacial till, may be designed to exert an ultimate bearing pressure of 450 kPa (Ultimate Limit States, ULS). The Serviceability Limit States (SLS) bearing pressure to limit footing settlement to 25 mm or less is 150 kPa (see Section 6.7, Limit States Resistance Factors and Serviceability, for assumed settlement criteria and maximum footing dimensions).

4. A minimum strip footing width of 500 mm is recommended. A minimum dimension of 1,000 mm for square and rectangular footings is recommended. If the subgrade soil at the design footing elevation consists of soft/wet soil, the width of the footing should be increased by fifty (50) percent.

5. If the subgrade soil should be disturbed during excavation below the design depth, then the disturbed soil should be removed to an undisturbed, level surface. Fill, required to bring the excavation to footing elevation should be fillcrete or concrete.

6. It is recommended that a mud slab should be placed as soon as practical after cleaning to minimize the potential for disturbance of the sand and silt subgrade soils.

7. A representative of the Geotechnical Consultant should inspect the excavation prior to the installation of footings.

8. Footings should not be constructed on desiccated, frozen or wet subgrade soil.

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

10. The finished grade should be landscaped to ensure positive drainage away from the building.

6.5 Drilled, Cast-In-Place Concrete Piles

Drilled, cast-in-place, straight shaft concrete piles should be designed on the basis of skin friction only.

The ultimate (ULS) and serviceability (SLS) bearing pressures of the undisturbed soil have been presented in Table IV.

Resistance factors to reduce the provided ultimate bearing pressures to a value that is suitable for design have been presented in Section 6.7, Limit States Resistance Factors and Serviceability.

TABLE IV. SKIN FRICTION BEARING PRESSURES (DRILLED PILES)

Zone (metres)	Skin Friction Bearing Pressures (kPa)	
	ULS	SLS
0 to 2	0	0
Below 2	75	30

<u>Notes:</u>

1. To minimize frost heave potential, skin friction piles should be extended to a minimum depth of 6 metres below finished ground surface.

2. Piles should be reinforced.

3. A minimum pile diameter of 400 mm is recommended for the primary structural loads.

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

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

6.6 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., skin friction) 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 ultimate (ULS) and serviceability (SLS) bearing pressures of the undisturbed soil have been presented in Tables V and VI. Resistance factors to reduce the provided ultimate bearing pressures to a value that is suitable for design have been presented in Section 6.7, Limit States Resistance Factors and Serviceability.

TABLE V. SKIN FRICTION BEARING PRESSURES (SCREW PILES)

Zone (metres)	Skin Friction Bearing Pressure (kPa)	
	ULS	SLS
0 to 2	0	0
Below 2	40	15

TABLE VI. END BEARING PRESSURES (SCREW PILES)

Depth (metres)	End Bearing Pressure (kPa)	
	ULS	SLS
3 to 5	650	260
Below 5	1,000	400

Notes:

1. The minimum embedment depth of the uppermost helix for multi-helix piles should be ≥ 3m 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 4.5 metres below grade or H/D = 5 (whichever is greater).

3. When determining the compressive skin friction 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 skin friction should be increased to 2D above the uppermost helix.

4. 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.

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

6. 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.

7. 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.

8. Screw piles should be designed on the basis of conventional static analysis using the provided bearing pressures presented in Tables V and VI. Installation torque should be used for monitoring purposes only and not to determine pile capacity.

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

6.7 Limit States Resistance Factors and Serviceability

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.

A further discussion of the limit states design method is described in the Canadian Foundation Engineering Manual (CFEM, 2006) and the National Building Code of Canada (NBCC, 2010).

As per NBCC - 2010, the following resistance factors may be applied to the ultimate bearing pressures to obtain the factored geotechnical resistance corresponding to ultimate limit states (ULS).

- Shallow foundations:
  - Compressive Resistance, $\Phi = 0.5$
  - Horizontal Load Resistance (cohesion/Adhesion), $\Phi = 0.6$
- Deep foundations:
  - Compressive Resistance, $\Phi = 0.4$
  - Tensile Resistance, $\Phi = 0.3$

For Limit States Design (LSD), a settlement analysis of the foundation must also be evaluated to ensure the structure is not negatively impacted by excessive settlement at the design load. This is also known as Serviceability Limit States (SLS) when designing on the basis of LSD.

With respect to a footing foundation at this site, the SLS bearing capacity is typically defined as bearing required for a settlement of 25 mm. In this case, a maximum footing size of 2 metres for a square and rectangular footings and 1 metre for strip footings was assumed. If the foundation is designed with a larger footing size than what is stated above, PMEL should re-evaluate the recommended SLS bearing capacity.

With respect to SLS and deep foundation design, provided the piles are designed using the resistance factors presented above and good construction practices are followed, the amount of settlement of a deep foundation at the design load will be small and within tolerable limits (within the range of 10 to 15 mm).

Piles designed on the basis of ULS or SLS bearing pressures will undergo the same or similar level of performance.

6.8 Floor Slabs

6.8.1 At-Grade Floor Slabs

The following minimum provisions should be incorporated into 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. To provide a level working surface and uniform subgrade support, provide a minimum of 150 mm of crushed, granular base course beneath the underside of the slab.

2. Level and compact the upper 150 mm of subgrade soil to 96 percent of standard Proctor density at optimum moisture content.

3. Excavate soft subgrade areas and replace with suitable, non-expansive fill, placed and compacted to 96 percent of standard Proctor density at optimum moisture content.

4. Subgrade fill, if required, should consist of imported 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.

5. The excavating/hauling equipment should not be allowed to travel on the prepared subgrade. The use of light equipment may be required for moisture conditioning, levelling and compaction of the uppermost 150 mm of the subgrade at final design elevation.

6. Do not allow the subgrade soil to dry out. Cover the prepared subgrade soil with non-expansive fill as soon as practical after preparation.

7. All granular fill above the design subgrade elevation should be placed in thin lifts (150 mm loose) and compacted to 98 percent of standard Proctor density at optimum moisture content.

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

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

10. Provide positive site drainage away from the proposed Building. Extend downspouts at least 3 metres away from the foundation.

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

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

6.8.2 Basement Floor Slabs

The minimum recommended provisions presented in Section 6.7.1 and the following additional provisions should be incorporated into the design of heated basement floor slabs.

1. Over-excavate the subgrade soil to allow for the placement of 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). Place a non-woven geotextile (see Section 6.8 for specifications) on the prepared subgrade. The drainage aggregate should meet the following gradation requirements.

Sieve Designation	Percent Passing
25.0 mm	100
9.5 mm	50 - 95
4.75 mm	35 - 70
2.00 mm	20 - 45
0.425 mm	0 - 20
0.150 mm	0 - 8
0.071 mm	0 - 3

2. Separate the slab from the fill by means of a polyethylene vapour barrier. Care should be taken during and following installation to minimize damaging the vapour barrier.

3. 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.

6.9 Basement Walls

Subsurface foundation walls should be designed to resist lateral earth pressure exerted by the backfill as well as the horizontal pressure induced by any surcharge loading. The lateral earth pressure may be calculated on the basis of an equivalent fluid pressure distribution of 9.5 kN/m^3 for drained conditions (i.e., perforated drainage pipe drainage system and clean, free-draining backfill as discussed below) and 15.7 kN/m^3 for walls backfilled with the existing subgrade soils.

The lateral earth pressure loading of 9.5 kN/m^3 assumes that the backfill will be free-draining, uniformly placed around the structure and lightly compacted, and, a perforated drainage pipe will be installed alongside the foundation walls with the invert elevation at or below the base of the foundation. 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 soils 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 (refer to Section 6.8.2 for gradation) above the invert of the drainage pipe. In the zone 300 mm above the invert of the drainage pipe and extending to within 500 mm of ground surface, clean, free-draining granular material with less than 5 percent material finer than the 0.071 mm sieve size should be used. The uppermost 500 mm should consist of clay or other low permeability material.

6.10 Grade Beams

The grade beams should be reinforced at both top and bottom throughout their entire length. Grade beams should be constructed to allow for a minimum of 100 mm of net void space between the underside of the grade beam and the subgrade soil.

6.11 Foundation Concrete

The results of water soluble sulphate testing on soil samples recovered during the field investigation have been summarized in Table VII.

TABLE VII. WATER SOLUBLE SULPHATE TEST RESULTS

Test Hole No.	Depth (m)	Soil Type	Water Soluble Sulphate (%)	Class of Exposure	Degree of Sulphate Exposure
17-4	1.5	Glacial Till	0.092	--	Negligible
17-4	6.0	Glacial Till	0.630	S-2	Severe

An examination of Table VII revealed that the measured sulphate contents were 0.092 and 0.63 percent, respectively, which is considered negligible to severe in terms of potential degree of sulphate attack. As such, sulphate resistant cement (Exposure Class S-2) should be used for all foundation concrete in contact with the soil.

All concrete at this site should be manufactured in accordance with current CSA standards. Grade-supported foundations supported entirely on engineered granular fill (i.e., negligible rating) may be designed utilizing General Use (CSA Symbol GU) cement.

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

6.12 Roadways

The following minimum recommendations should be incorporated into the design of the roadways for the proposed subdivision.

1. Prepare the site in accordance with Section 5.2, Site Preparation.

2. Scarify the upper 150 mm of the subgrade soil, moisture condition and compact to 96 percent of standard Proctor density at optimum moisture content. Proof roll the prepared subgrade with heavy wheeled equipment to detect soft areas. 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. Site specific design recommendations can be provided by the Geotechnical Consultant upon request.

3. As a subgrade support, the CBR (California Bearing Ratio) rating of the compacted subgrade soil should be in the order of 3. Based on the CBR rating, the following gravel structures have been presented in Table VIII.

TABLE VIII. THICKNESS DESIGN FOR ROADWAY STRUCTURES

Road Structure	Light to Medium Truck/Passenger Vehicle Traffic (mm)		Heavy Truck Traffic (mm)	
	Asphalt	**Gravelled**	**Asphalt**	**Gravelled**
Asphalt Concrete	65	--	100	--
Granular Base	150	150	150	150
Granular Sub-Base	150	300	250	450
Prepared Subgrade	(150)	(150)	(150)	(150)
Geotextile	As Required	As Required	As Required	As Required
Total Thickness	**365**	**450**	**500**	**600**

4. Subgrade fill, if required, may consist of imported granular material or non-expansive fine-grained soil. Subgrade fill should be placed in thin lifts (150 mm loose, maximum) and compacted to 96 percent of standard Proctor density at optimum moisture content.

5. All granular fill placed above the subgrade elevation 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 and sub-base course material should meet the aggregate gradation requirements presented in Table IX.

TABLE IX. AGGREGATE GRADATION REQUIREMENTS

Grain Size (mm)	Percent Passing	
	Base Course	Sub-Base Course
50.0	--	100
18.0	100	--
12.5	75-100	--
5.0	50-75	--
2.0	32-52	0-80
0.900	20-35	--
0.400	15-25	0-45
0.160	8-15	0-20
0.080	6-11	0-6
Plasticity Index (%)	6 max	6 max.
% Fracture (min.)	50	--

6. Positive surface drainage is recommended to reduce the potential for moisture infiltration through the pavement structure and subgrade softening.

7. Surface water should be prevented from seeping back under the outer edges of the pavement structure.

8. Depending on final grade, ditches and culverts should be provided where necessary to provide adequate site drainage. The invert elevation of the ditch should preferably be in the order of 1 metre below the edge of the roadway (a lesser depth could be accepted in areas to satisfy lateral constraints).

9. Embankment side slopes should be no steeper than 2.0 Horizontal to 1.0 Vertical (2H:1V). Backslopes of 3H:1V are preferred.

10. Periodic maintenance such as surface grading (to provide a level riding surface) is recommended.

7.0 LIMITATIONS

The presentation of the summary of the field drill logs and design recommendations and slope stability analysis has been completed as authorized. Five, 150 mm diameter test holes were dry drilled using our truck-mounted 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 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 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 Prairie's Edge Development Corporation and their agents for specific application to Phase I of the proposed new Subdivision located within 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 is 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 the report fulfils your requirements for this project. Please call if you have any questions or additional information is required.

P. MACHIBRODA ENGINEERING LTD.

PROFESSIONAL ENGINEER
G.A.A. BAXTER
MEMBER 15402
2017 11 06
YR. MO. 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	2017-11-06

P. MACHIBRODA
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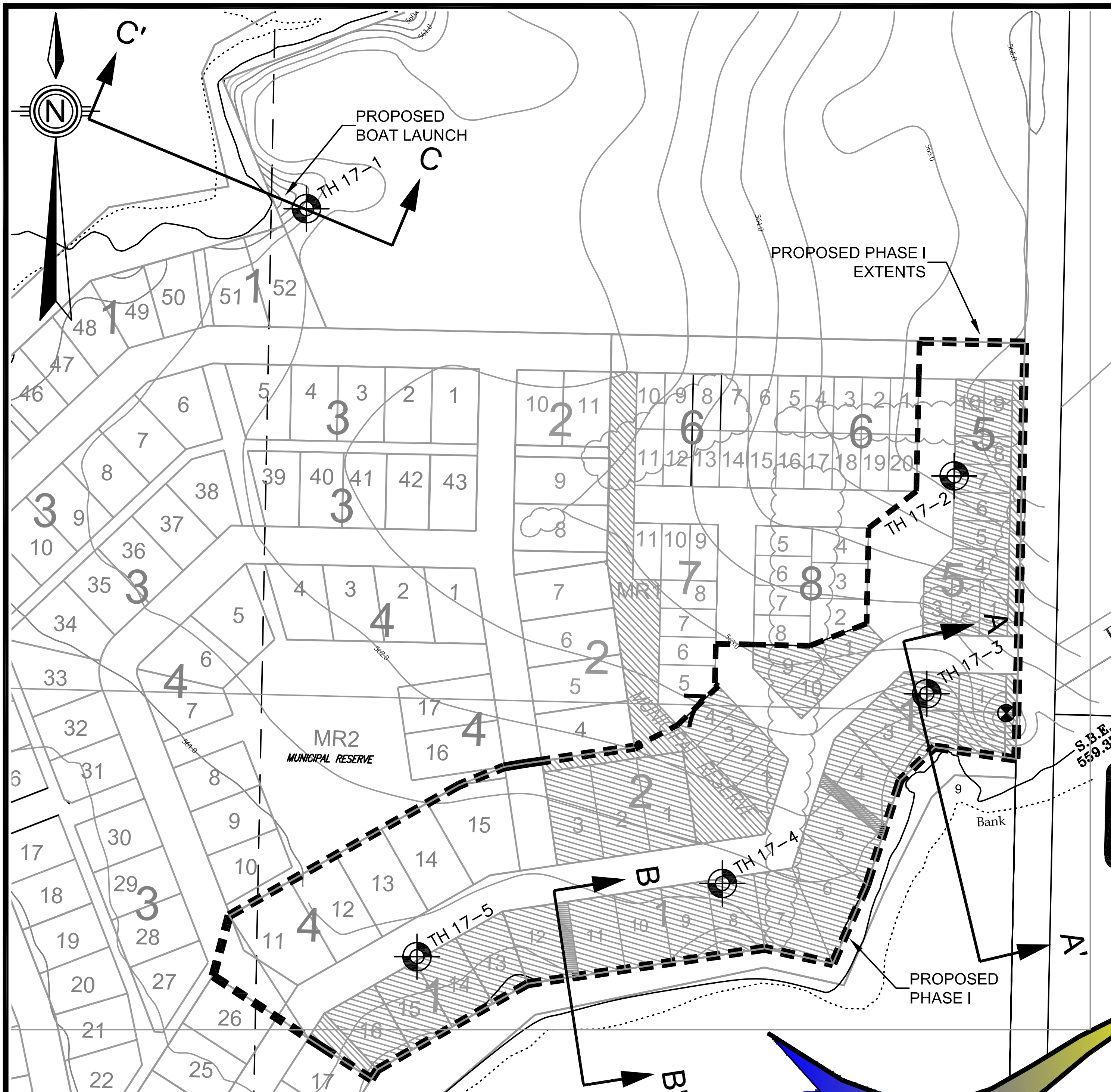
CONSULTING
GEOTECHNICAL/GEOENVIRONMENTAL
ENGINEERS

DRAWINGS



NOTE:

1. THIS DRAWING IS FOR CONCEPTUAL PURPOSES ONLY. ACTUAL LOCATIONS MAY VARY AND NOT ALL STRUCTURES ARE SHOWN.
2. BENCHMARK: FOUND IRON PIN LEGAL SURVEY MARKER = ORTHOMETRIC ELEVATION OF 562.687 m WAS PROVIDED BY ASSOCIATED ENGINEERING FOR THE TOP OF PIN.

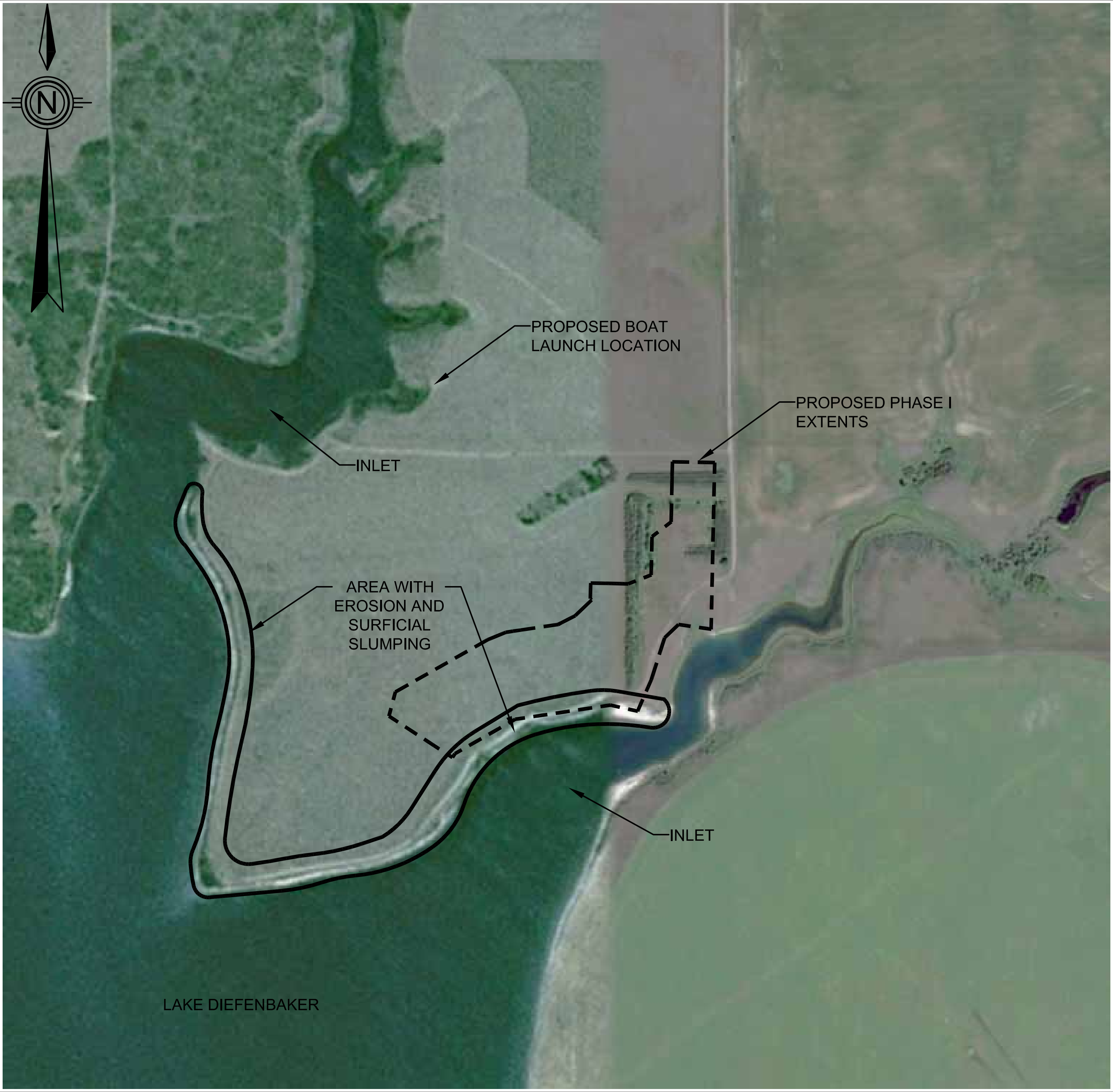
LEGEND	–PMEL TEST HOLE (PIEZOMETER INSTALLED)	–BENCHMARK

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 PART OF 27-25-6-W3M, RM OF LOREBURN No. 254, SK		
APPROVED BY: GB	DRAWN BY: SD	DRAWING NUMBER: 13147-1
DATE: OCTOBER, 2017	SCALE: NOT TO SCALE	



NOTE:
1. THIS DRAWING IS FOR CONCEPTUAL PURPOSES ONLY. ACTUAL LOCATIONS MAY VARY AND NOT ALL STRUCTURES ARE SHOWN.
2. BENCHMARK: FOUND IRON PIN LEGAL SURVEY MOLAR = ORTHOMETRIC ELEVATION OF 562.687 m WAS PROVIDED BY ASSOCIATED ENGINEERING FOR THE TOP OF PIN.

LEGEND

- PMEL TEST HOLE (PIEZOMETER INSTALLED)
- BENCHMARK

CONSULTING GEOENVIRONMENTAL GEOTECHNICAL ENGINEERS

P. MACHIBRODA ENGINEERING LTD.

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

DRAWING TITLE:

AERIAL PLAN

PROJECT:

PROPOSED SUBDIVISION
PART OF 27-25-6-W3M, RM OF LOREBURN No. 254, SK

APPROVED BY:	DRAWN BY:	DRAWING NUMBER:
GB	SD	13147-1A
DATE: OCTOBER, 2017	SCALE: NOT TO SCALE	

PIEZO. ELEV.= 562.9 m

TEST HOLE 17-1

DEPTH (m)

ELEV: 561.9 m

N	U (pp)	γ_w	Pw	Lw	w
					7.7
			13	35	9.5
					10.6
					10.7
18		21.6			12.3
			16	39	16.3
23		21.2			16.4
					14.9
					17.4
					15.7

BENTONITE SEAL

50 mm diam. SCH 40, PVC RISER PIPE

CUTTINGS

555.7 m OCT 30/17

TOPSOIL, black, organics, rootlets.

GLACIAL TILL, clay, some silt, some sand, trace gravel, stiff, medium plastic, damp, brown, oxide stained.
—moist below 500 mm.

—very stiff below 1.6 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

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: PART OF 27-25-6-W3M, RM OF LOREBURN No. 254, SK

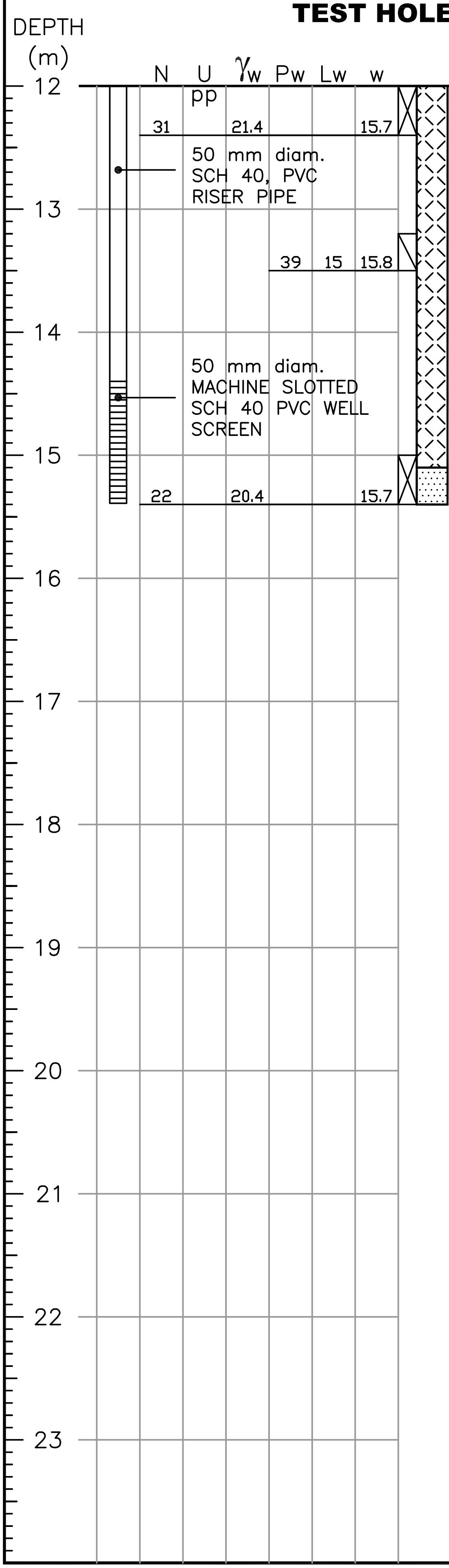
NORTHING:

EASTING:

DATE DRILLED: OCT 10/17

DRAWING NUMBER: 13147-2

TEST HOLE 17-1



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

SAND, trace silt, dense, poorly graded, fine to coarse grained, wet, brown, seepage, sloughing.

NOTE:
1. Test Hole open with trace water 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: PART OF 27-25-6-W3M, RM OF LOREBURN No. 254, SK

NORTHING:

EASTING:

DATE DRILLED: OCT 10/17

DRAWING NUMBER: 13147-2A

PIEZO. ELEV.= 566.2 m

TEST HOLE 17-2

DEPTH (m)

N U γ_w Pw Lw w ELEV: 565.2 m

Depth (m)		N	U	γ_w	Pw	Lw	w
0	pp						
	BENTONITE SEAL				14	26	8.1
1							13.2
2		11		21.1			13.7
	50 mm diam. SCH 40, PVC RISER PIPE						
3							14.4
	CUTTINGS						
4	560.9 m OCT 30/17						
5		13					17.5
6							16.1
7							16.9
			2.5				
8	50 mm diam. MACHINE SLOTTED SCH 40 PVC WELL SCREEN						
	SAND PACKED ANNULUS						
9							16.6
10							
11							

TOPSOIL, black, organics, rootlets.

SILT, some clay, trace sand, firm, low plastic, damp, brown.

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

—very stiff below 6.1 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.

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FIELD DRILL LOG AND SOIL TEST RESULTS

PROJECT: PROPOSED SUBDIVISION

LOCATION: PART OF 27-25-6-W3M, RM OF LOREBURN No. 254, SK

NORTHING: **EASTING:**

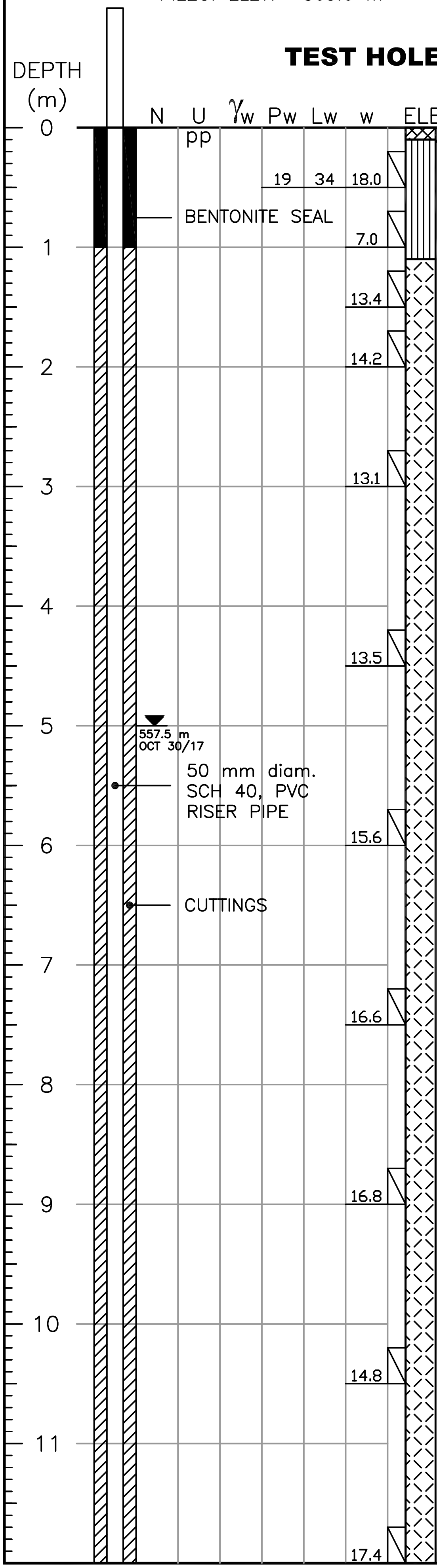
DATE DRILLED: OCT 10/17

DRAWING NUMBER: 13147-3

PIEZO. ELEV.= 563.6 m

TEST HOLE 17-3

ELEV: 562.5 m



TOPSOIL, black, organics, rootlets.

SILT, clayey, trace sand, firm, medium plastic, moist, brown, oxide stained.

–some clay, low plastic, damp below 700 mm.

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

–very stiff below 4.6 m.

–trace seepage at 10.6 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

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

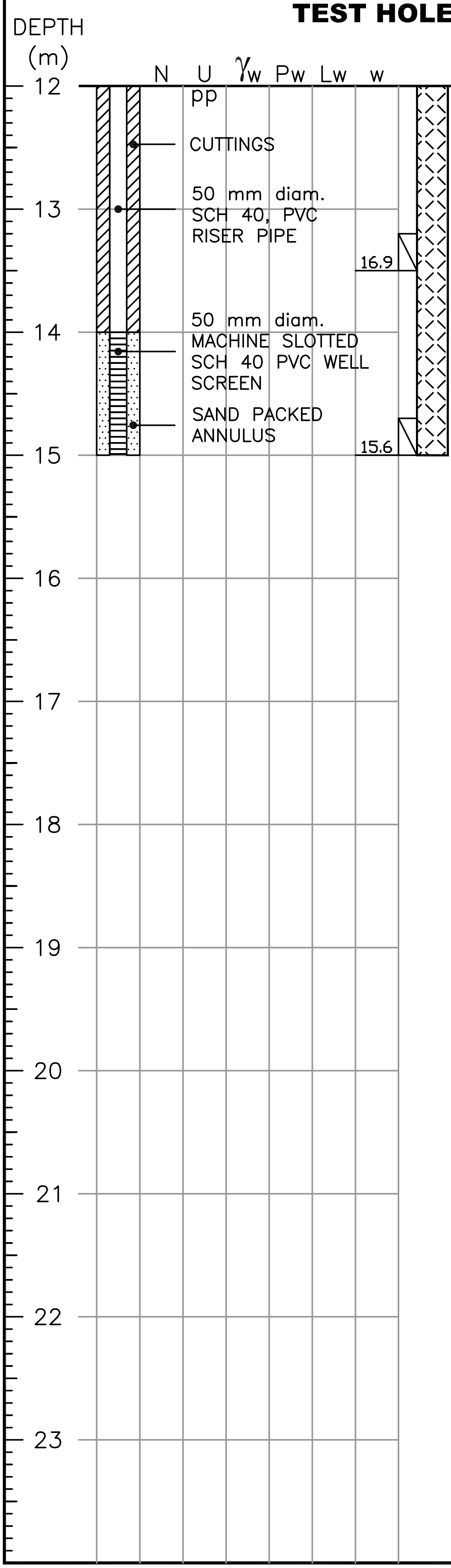
LOCATION:

PART OF 27-25-6-W3M,
RM OF LOREBURN No. 254, SK

NORTHING: **EASTING:**

DATE DRILLED:	**DRAWING NUMBER:**
OCT 10/17	13147-4

TEST HOLE 17-3



GLACIAL TILL, clay, some silt, some sand, trace gravel, hard, medium plastic, moist, grey, gypsum crystals.

NOTE:
1. Test Hole open with trace water 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

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: PART OF 27-25-6-W3M, RM OF LOREBURN No. 254, SK

NORTHING:

EASTING:

DATE DRILLED: OCT 10/17

DRAWING NUMBER: 13147-4A

PIEZO. ELEV.= 562.6 m

TEST HOLE 17-4

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

N	U (pp)	γ_w	Pw	Lw	w
					14.4
					5.3
					8.5
					9.6
33		21.9			9.4
					13.6
					15.4
					17.0
28		21.1			17.7

BENTONITE SEAL

50 mm diam. SCH 40, PVC RISER PIPE

CUTTINGS

SAND PACKED ANNULUS

50 mm diam. MACHINE SLOTTED SCH 40 PVC WELL SCREEN

553.3 m OCT 30/17

ELEV: 561.5 m

TOPSOIL, black, organics, rootlets.

SILT, clayey, some sand, firm, medium plastic, moist, brown.
—some clay, low plastic, damp below 500 mm.

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

—very stiff to hard below 4.6 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: PART OF 27-25-6-W3M, RM OF LOREBURN No. 254, SK

NORTHING:

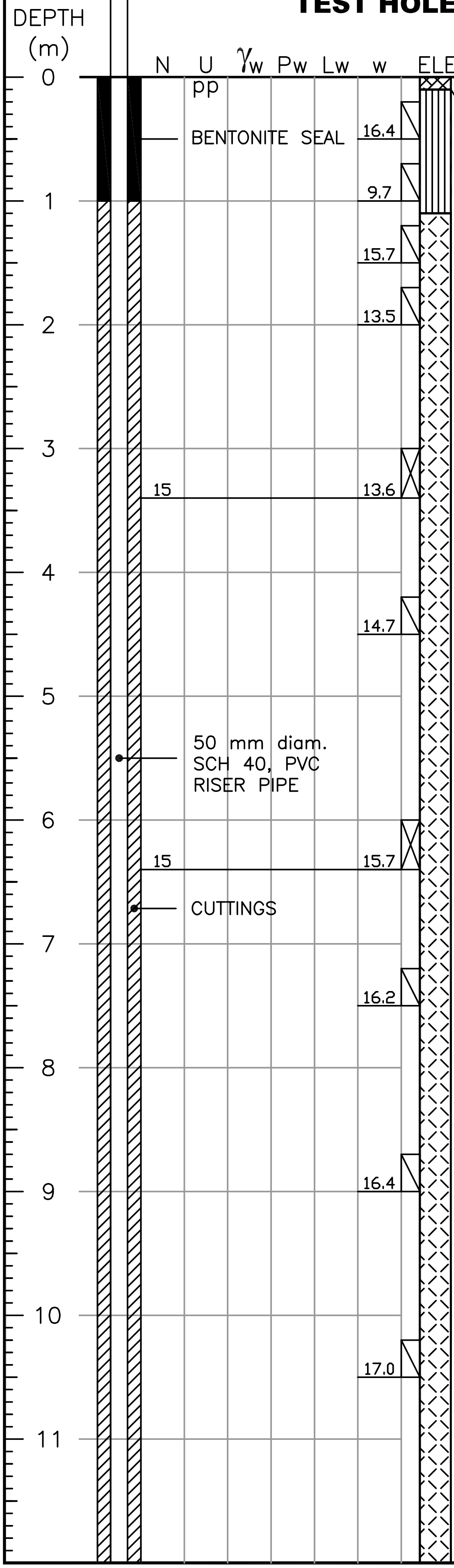
EASTING:

DATE DRILLED: OCT 10/17

DRAWING NUMBER: 13147-5

PIEZO. ELEV.= 561.4 m

TEST HOLE 17-5



TOPSOIL, black, organics, rootlets.

SILT, clayey, trace sand, firm, medium plastic, moist, brown.
–some clay, low plastic, damp below 500 mm.

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

–very stiff below 3.0 m.

CONTINUED ON NEXT PAGE

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

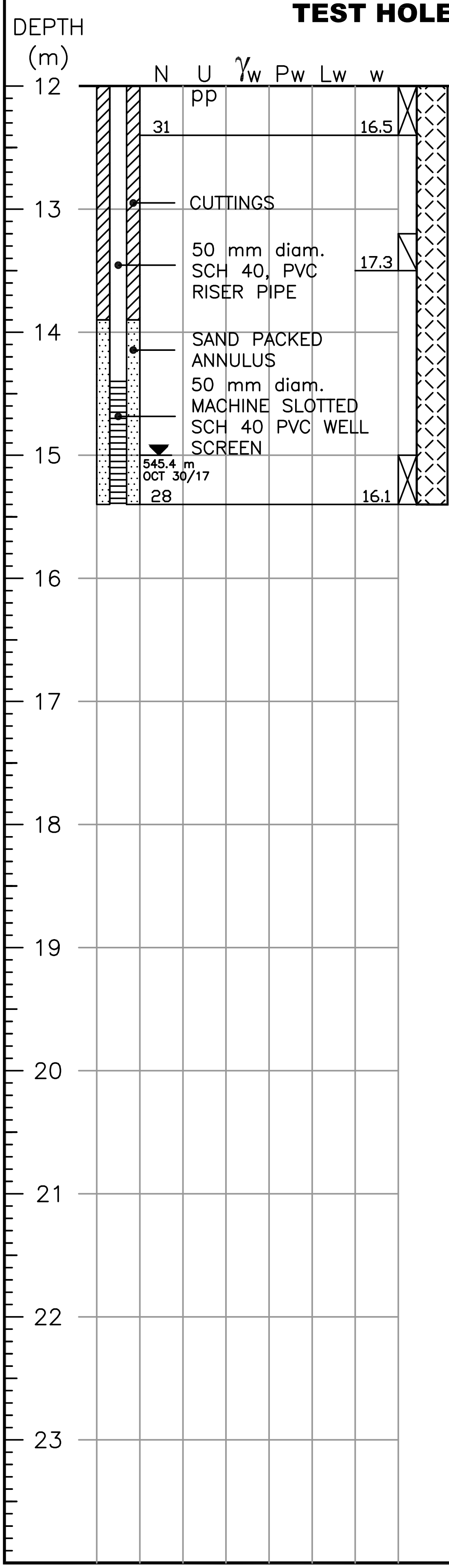
LOCATION: PART OF 27-25-6-W3M, RM OF LOREBURN No. 254, SK

NORTHING: **EASTING:**

DATE DRILLED: OCT 10/17

DRAWING NUMBER: 13147-6

TEST HOLE 17-5



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/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: PART OF 27-25-6-W3M, RM OF LOREBURN No. 254, SK

NORTHING:

EASTING:

DATE DRILLED: OCT 10/17

DRAWING NUMBER: 13147-6A

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

Clay – particles of size	< 0.002 mm
Silt – particles of size	0.002 – 0.060 mm
Sand – particles of size	0.06 – 2.0 mm
Gravel – particles of size	2.0 – 60 mm
Cobbles – particles of size	60 – 200 mm
Boulders – particles of size	>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.

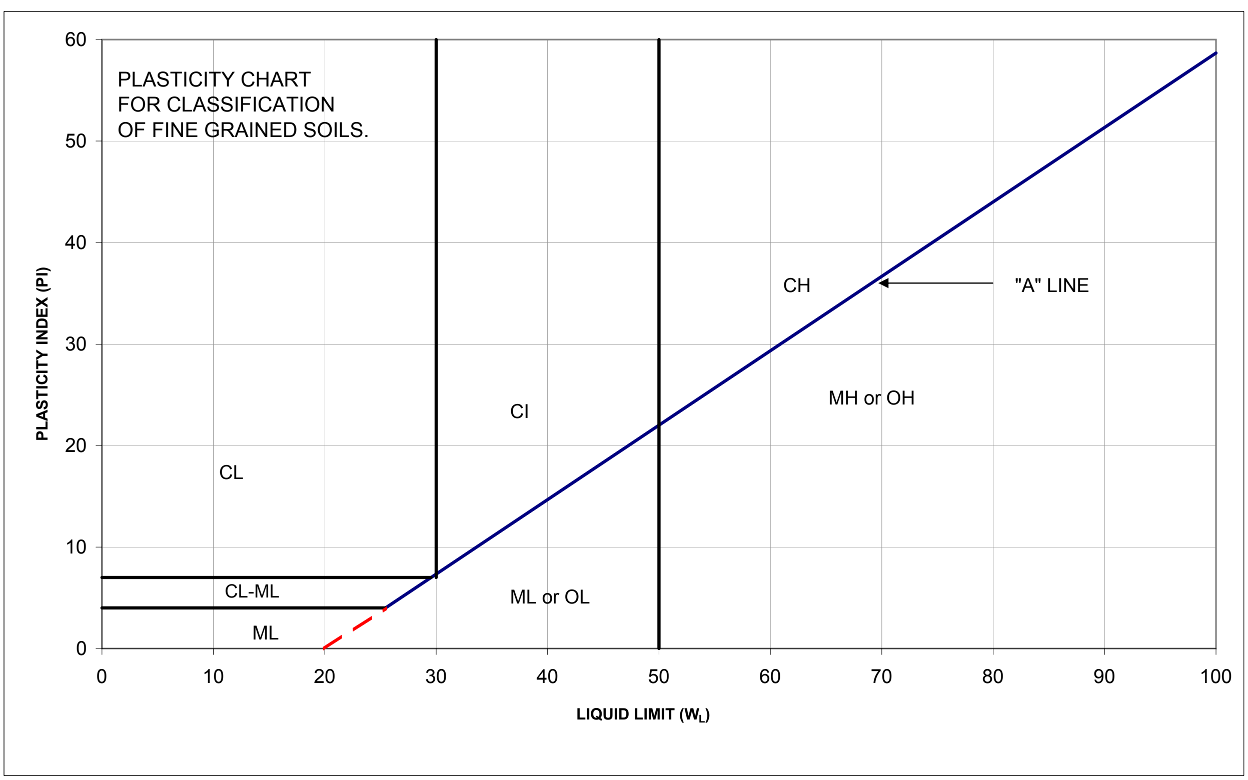
Organic Soils: Readily identified by colour, odour, spongy feel and frequently by fibrous texture.

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
LABORATORY TEST RESULTS

P. MACHIBRODA ENGINEERING LTD.

ASTM D422: PARTICLE SIZE ANALYSIS OF SOILS

Project: Proposed Subdivision

Location: Part of 27-25-6-W3M, RM of Loreburn No. 254

Project No.: 13147

Date Tested: October 23, 3017

Test Hole No.: 17-1

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	100
# 4	4.75	100
# 10	2	99
# 20	0.85	96
# 40	0.425	91.8
#60	0.25	86.3
# 100	0.15	79.5
# 200	0.075	67.0

Hydrometer Analysis:

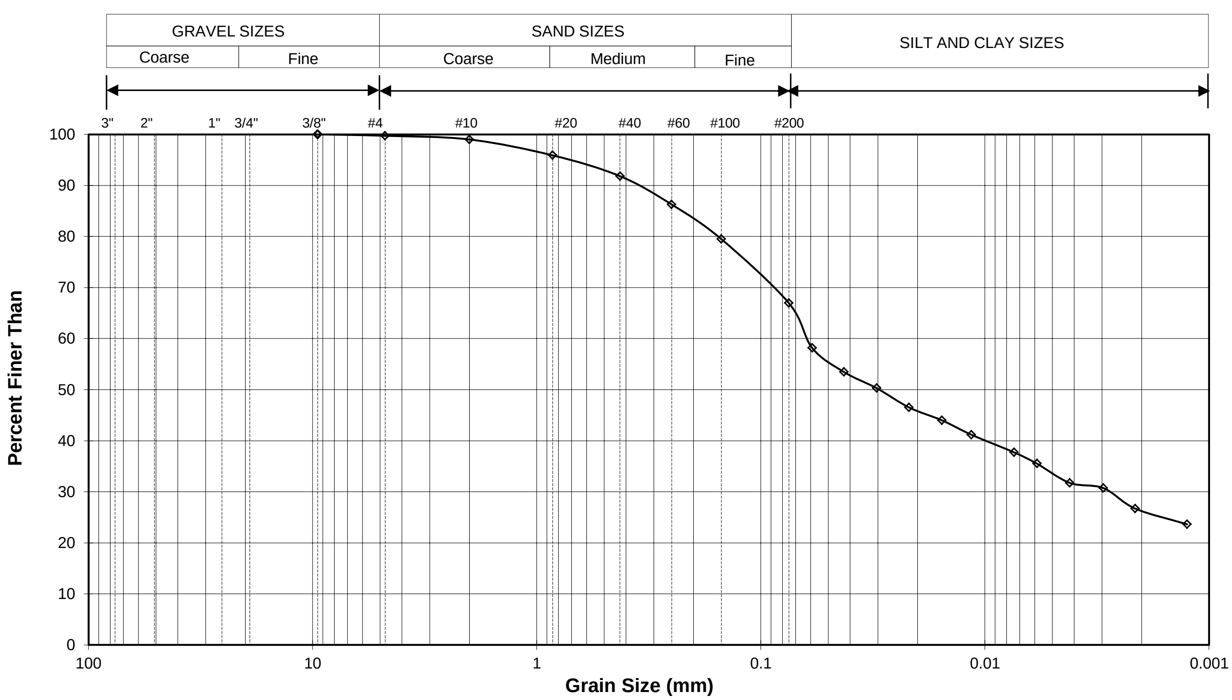
Dispersing Agent: *Sodium Hexametaphosphate*

Diameter mm	% Finer
0.0590	58.2
0.0425	53.4
0.0304	50.3
0.0218	46.5
0.0156	44.0
0.0115	41.2
0.0074	37.7
0.0059	35.5
0.0042	31.7
0.0030	30.7
0.0021	26.7
0.0013	23.6

Material Description:

% Gravel Sizes	% Sand Sizes	% Silt Sizes	% Clay Sizes
0	33	40	27

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 Schergevitch

P. MACHIBRODA ENGINEERING LTD.

ASTM D422: PARTICLE SIZE ANALYSIS OF SOILS

Project: Proposed Subdivision
Location: Part of 27-25-6-W3M, RM of Loreburn No. 254
Project No.: 13147
Date Tested: October 23, 3017
Test Hole No.: 17-1
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	99
3/8"	9.5	99
# 4	4.75	97
# 10	2	97
# 20	0.85	93
# 40	0.425	88.3
#60	0.25	81.8
# 100	0.15	74.2
# 200	0.075	63.2

Hydrometer Analysis:

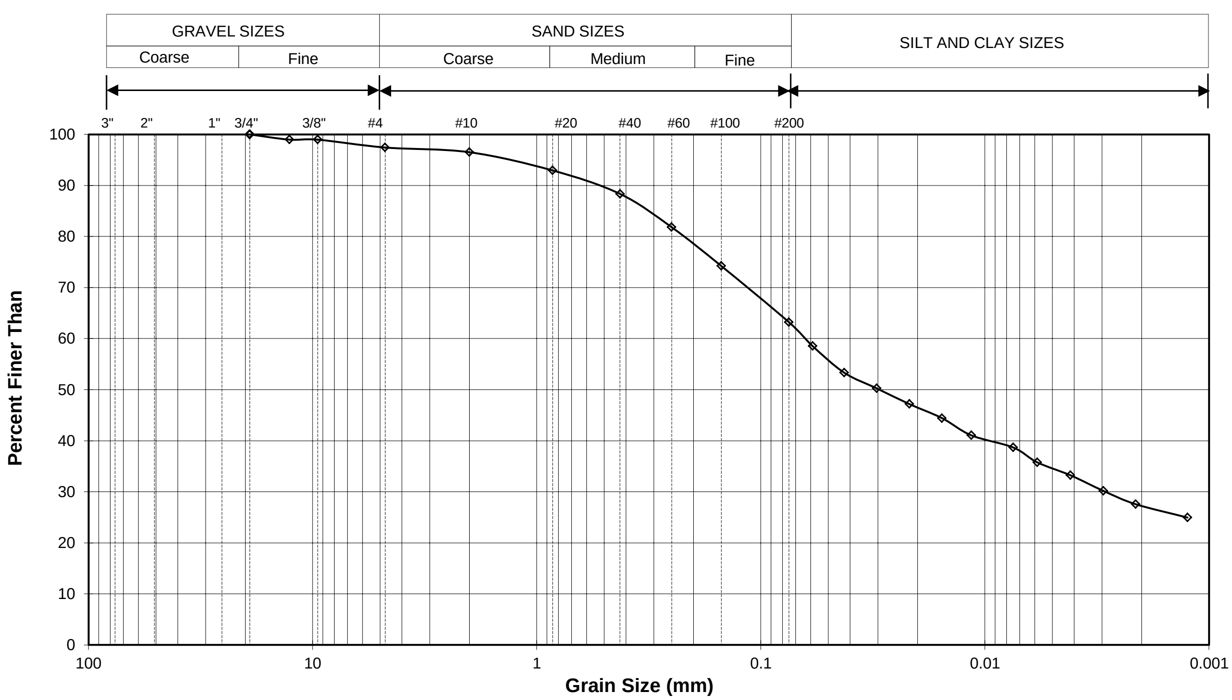
Dispersing Agent: *Sodium Hexametaphosphate*

Diameter mm	% Finer
0.0587	58.5
0.0424	53.3
0.0304	50.3
0.0217	47.2
0.0155	44.4
0.0115	41.1
0.0075	38.7
0.0058	35.8
0.0042	33.2
0.0030	30.2
0.0021	27.6
0.0012	25.0

Material Description:

% Gravel Sizes	% Sand Sizes	% Silt Sizes	% Clay Sizes
3	34	36	27

Remarks:



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Drawing No.

Appendix B-2

WE CERTIFY TESTING PROCEDURES ARE IN ACCORDANCE WITH ASTM D422 STANDARD
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PER Preston Schergevitch

P. MACHIBRODA ENGINEERING LTD.

ASTM C136: GRAIN SIZE ANALYSIS

Project: Proposed Subdivision

Location: Part of 27-25-6-W3M, RM of Loreburn No. 254

Project No.: 13147

Date Tested: October 23, 3017

Test Hole No: 17-1

Sample No.: 13

Depth: 15-15.4

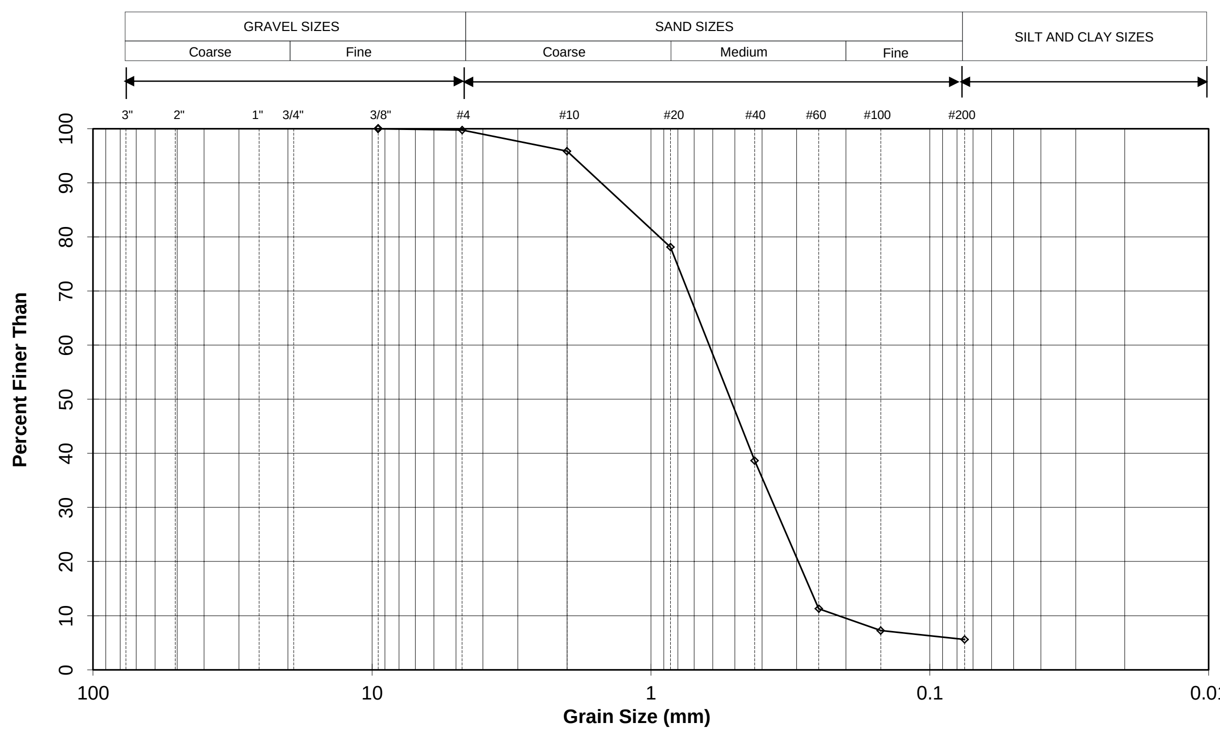
Sieve Analysis:

Sieve	Diameter mm	% Finer
3"	76.200	100
2.5"	63.500	100
2"	50.000	100
1.5"	37.500	100
1	25.000	100
0.75	19.000	100
0.5	12.500	100
0.375	9.500	100
4	4.750	100
10	2.000	96
20	0.850	78
40	0.425	39
60	0.250	11
100	0.150	7.3
200	0.075	5.6

Material Description:

% Gravel Sizes	% Sand Sizes	% Silt and Clay Sizes
	94	6

Remarks:



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DRAWING NO.

Appendix B-3

WE CERTIFY TESTING PROCEDURES ARE IN ACCORDANCE WITH ASTM C136 STANDARD
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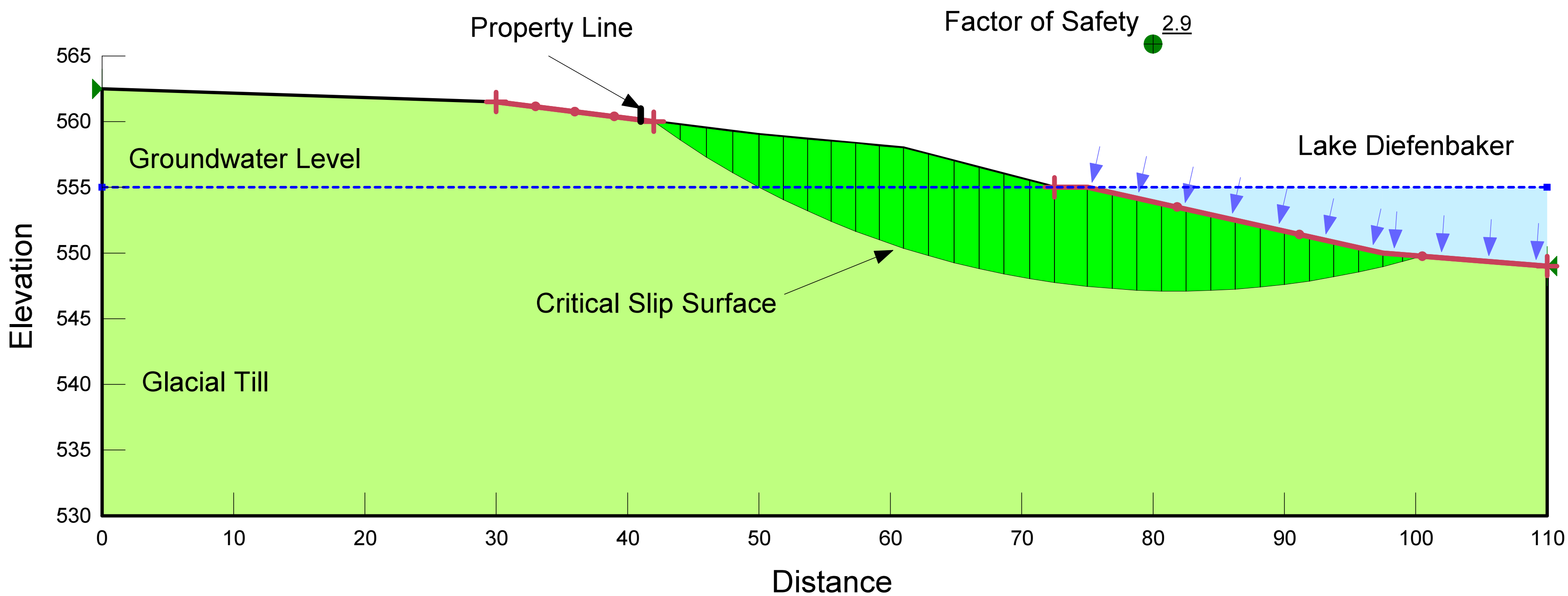
PER Preston Schergevitch

APPENDIX C
TYPICAL SLOPE STABILITY PLOTS

PMEL File No. 13147
Drawing No. Appendix C-1

SECTION A-A'



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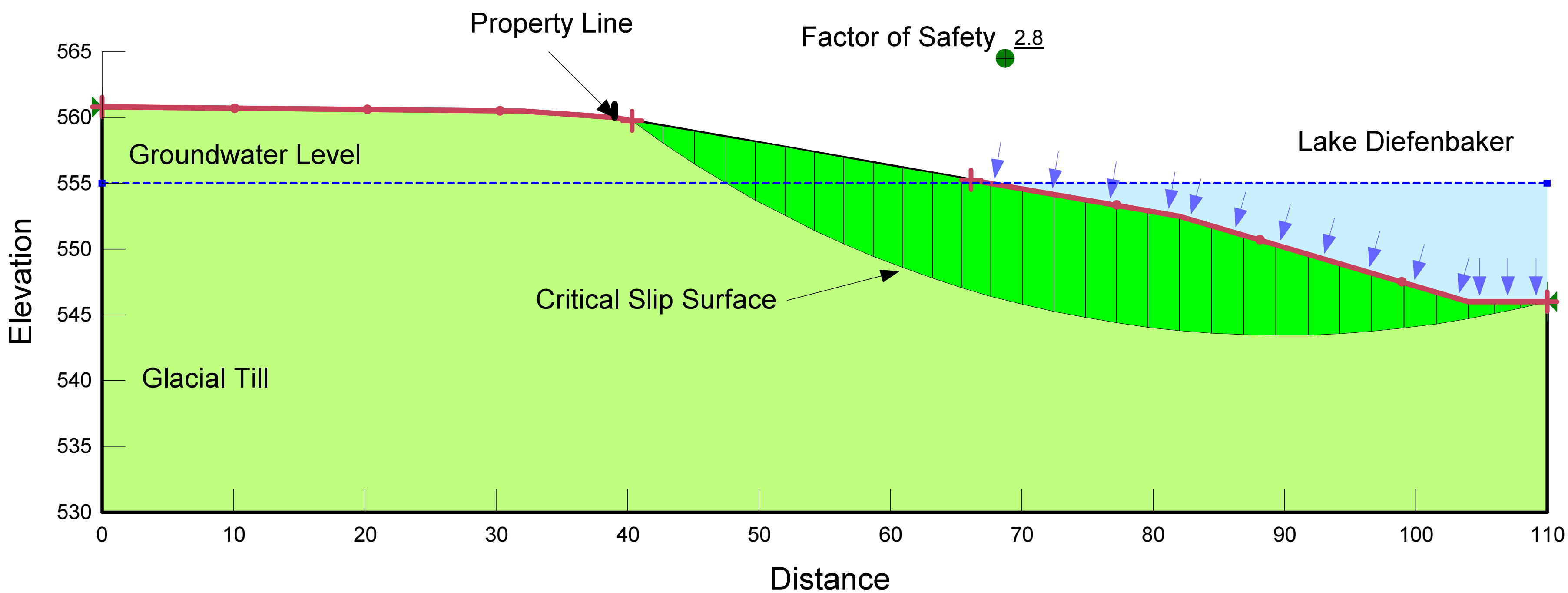
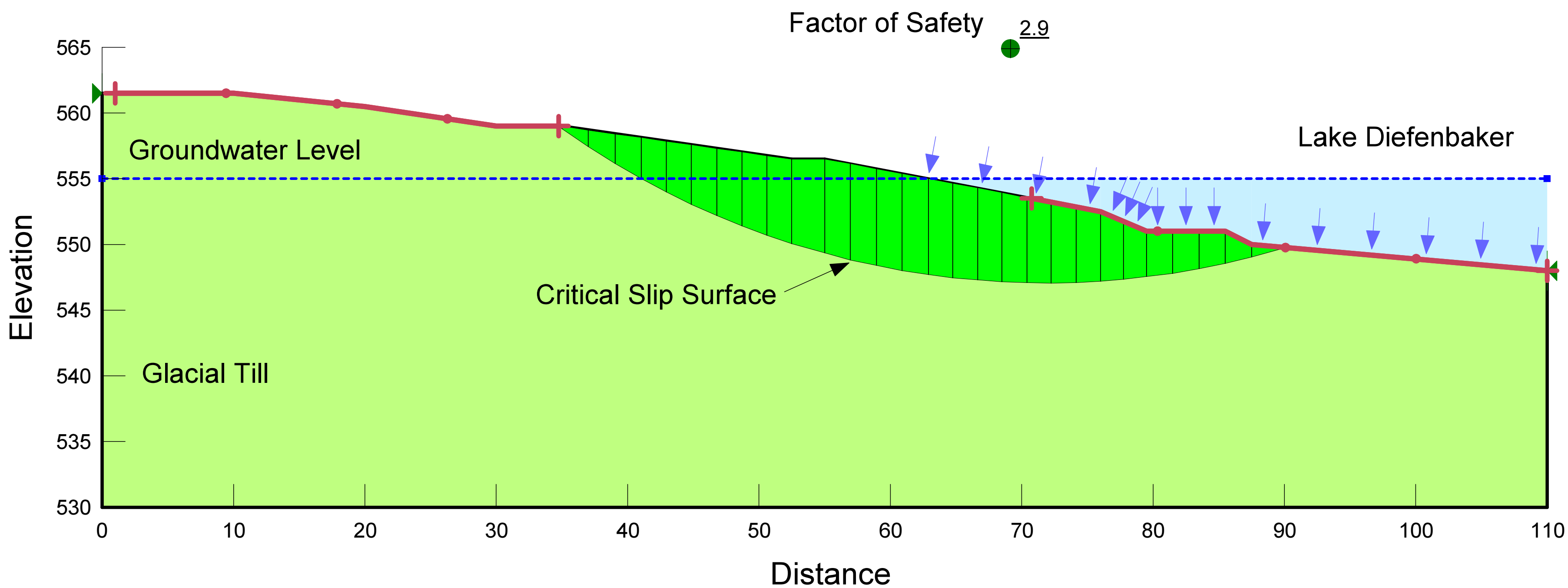
SECTION B-B'


Property Line
Factor of Safety 2.8
Lake Diefenbaker
Groundwater Level
Critical Slip Surface
Glacial Till
Elevation
565
560
555
550
545
540
535
530
0
10
20
30
40
50
60
70
80
90
100
110
Distance

PMEL File No. 13147

Drawing No. Appendix C-3

SECTION C-C'
BOAT LAUNCH AREA



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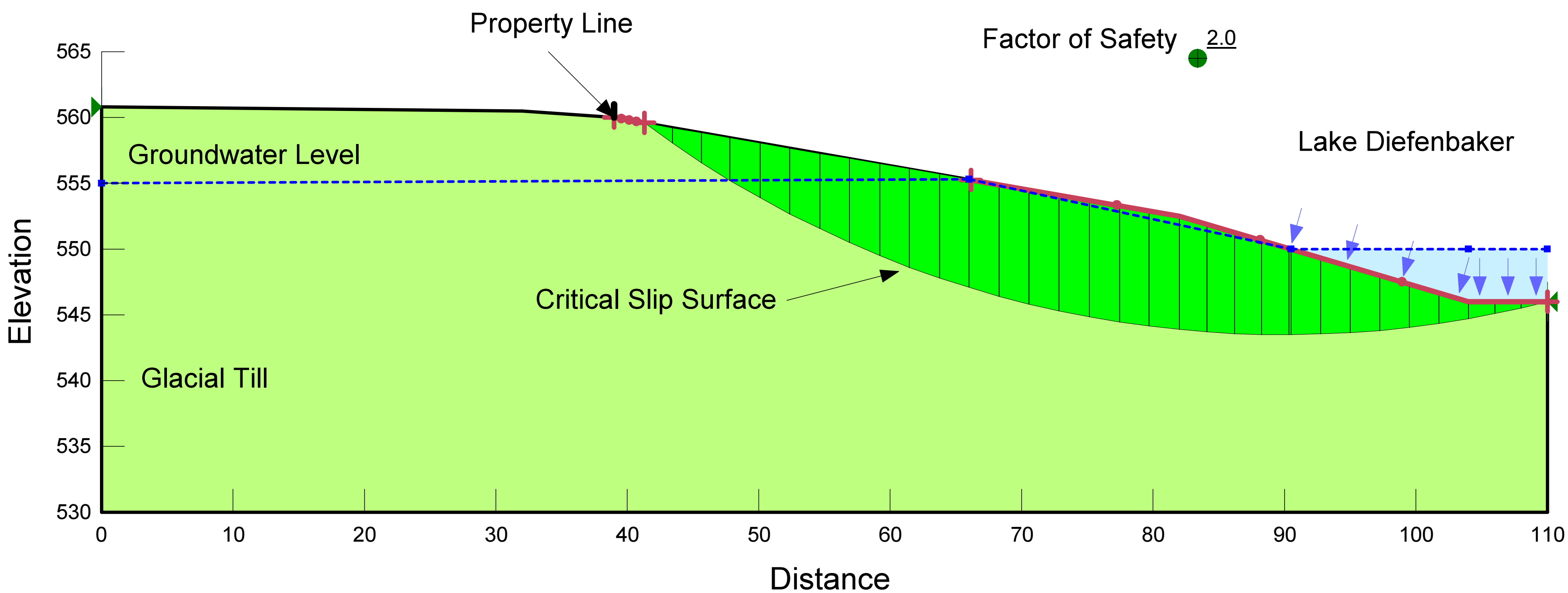
PMEL File No. 13147

Drawing No. Appendix C-4

SECTION B-B'
5 m Drop in Lake Water Level



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APPENDIX D
PHOTOGRAPHS

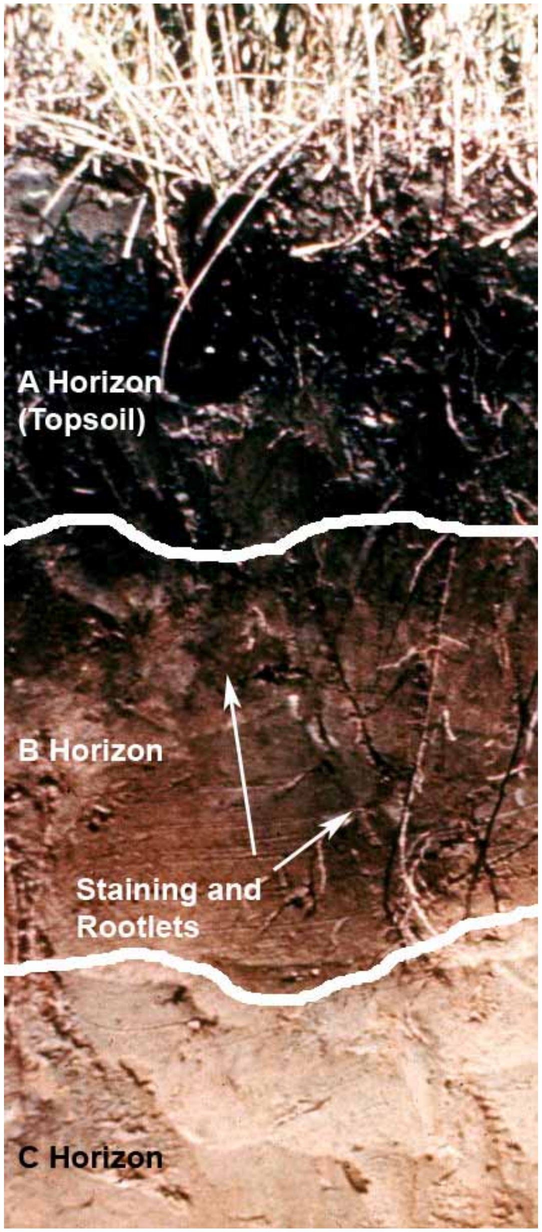
PHOTOGRAPH NO. 13147-01 Looking west from Legal Survey Maker at the northeast corner of SE-27-25-6-W2M.

PHOTOGRAPH NO. 13147-02 Surficial slumping and erosion along shoreline. Approximately at west end of Phase I

PHOTOGRAPH NO. 13147-03 Proposed Boat Launch area.

PHOTOGRAPH NO. 13147-04 Shoreline along west side of subject site (not part of Phase I).

APPENDIX E
TOPSOIL, ORGANIC MATTER AND ORGANICS



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.