

**P. MACHIBRODA
ENGINEERING
LTD.**

CONSULTING
GEOTECHNICAL
GEOENVIRONMENTAL
ENGINEERS AND
GEOSCIENTISTS

SASKATOON

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

PHONE:

(306) 665-8444

FAX:

(306) 652-2092

E-MAIL:

pmel.sk@machibroda.com

WEB:

www.machibroda.com

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**GEOTECHNICAL INVESTIGATION
PROPOSED FUEL STORAGE TANK PAD & PROPANE
DISPENSER FILL STATION
SASKATCHEWAN PENITENTIARY
PRINCE ALBERT, SASKATCHEWAN
PMEL FILE NO. S14-8725
APRIL 11, 2014**

PREPARED FOR:

**WSP CANADA INC.
1600 BUFFALO PLACE
WINNIPEG, SASKATCHEWAN
R3T 6B8**

ATTENTION: ALANA GAUTHIER, P. ENG.

TABLE OF CONTENTS

	Page
1.0 INTRODUCTION.....	1
2.0 FIELD INVESTIGATION	1
2.1 Field Drilling.....	1
2.2 Piezocone Penetration Testing	2
3.0 FIELD DRILL LOGS	2
3.1 Soil Profile.....	3
3.2 Groundwater Conditions, Sloughing.....	3
3.3 Cobbles and Boulders	3
4.0 LABORATORY ANALYSIS	3
5.0 DESIGN RECOMMENDATIONS	4
5.1 Design Considerations	4
5.2 Site Preparation	5
5.3 Perimeter Thickened Edge Raft Foundation	6
5.4 Driven, Treated Timber Piles.....	8
5.5 Helical Screw Piles.....	10
5.6 Factor of Safety/Resistance Factors.....	11
5.7 Grade-Supported Floor Slabs	12
5.8 Foundation Concrete.....	13
6.0 LIMITATIONS	14

LIST OF TABLES

Table I	Skin Friction Bearing Pressures (Driven, Timber Piles)
Table II	Termination Criteria (Driven, Timber Piles)
Table III	Skin Friction Bearing Pressures (Screw Piles)
Table IV	End Bearing Pressure (Screw Piles)
Table V	Water Soluble Sulphate Test Results

LIST OF DRAWINGS

S14-8725-1	Site Plan – Test Hole Locations
S14-8725-2	Field Drill Log and Soil Test Results
S14-8725-3	Field Drill Log and Soil Test Results

LIST OF APPENDICES

Appendix A	Explanation of Terms on Test Hole Logs
Appendix B	Topsoil, Organic Matter and Organics
Appendix C	CPTu Plots
Appendix D	Laboratory Results

1.0 INTRODUCTION

The following report has been prepared on the subsurface soil conditions existing at the site of the proposed Fuel Storage Tank Pad & Propane Dispenser Fill Station to be constructed at the Saskatchewan Penitentiary in Prince Albert, Saskatchewan. The site is located approximately 150 metres south of 15th Street West.

The terms of reference for this investigation were presented in P. Machibroda Engineering Ltd. (PMEL) Proposal No. 0301-7798REV, dated March 3, 2014.

The field test drilling and soil sampling were conducted on March 18, 2014.

2.0 FIELD INVESTIGATION

2.1 Field Drilling

Two test holes, located as shown on the Site Plan, Drawing No. S14-8725-1, were dry drilled using our truck-mounted continuous flight, solid stem auger drill rig. The test holes were 150 mm in diameter and extended to depths of 9 to 12 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 were collected during test drilling and 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.

2.2 Piezocone Penetration Testing

One piezocone penetration test (CPTu) was conducted during the field investigation and extended to a depth of 6.5 metres below existing grade. The CPTu location is shown on the Site Plan, Drawing No. S14-8725-1.

The piezocone penetration test consisted of pushing a cone, on the end of a series of rods, into the ground at a constant rate and continuous measurements were made of the resistance to penetration of the cone. Local side friction resistance measurements were also made on a friction sleeve during penetration. Pore-water pressure response generated from the advancement of the cone into the soil was measured via a pore pressure filter located directly behind the cone tip. The piezocone tip had an apex angle of 60° and a 10 cm² base area. The friction sleeve had a perimeter area of 150 cm².

The equipment and procedures for conducting the cone penetration testing were undertaken in accordance with ASTM D-5778, "Standard Test Method for Performing Electronic Friction Cone and Piezocone Testing of Soils".

The test plot generated during the cone sounding have been presented in Appendix C.

3.0 FIELD DRILL LOGS

The field drill logs recorded during test drilling have been shown plotted on Drawing Nos. S14-8725-2 and 3.

The ground surface elevation at each Test Hole location was referenced to top of the existing concrete floor slab, located approximately as shown on the Site Plan, Drawing No. S14-8725-1. A datum elevation of 100.00 metres was assumed for the top of the concrete floor slab.

3.1 Soil Profile

The general subgrade soil conditions consisted of granular fill (approximately 150 to 400 mm) overlying sand (extending to depths of about 5.4 to 5.7 metres), followed by clay, extending to a depth of at least 12 metres below existing grade, the maximum depth explored with our Test Holes at this site. A layer of clay was encountered in Test Hole No. 14-1 at approximately 0.2 metres below grade extending to a depth of about 1.3 metres.

3.2 Groundwater Conditions, Sloughing

Extensive groundwater seepage and sloughing conditions were encountered in the sand stratum during test drilling. The depths at which groundwater seepage and sloughing conditions were encountered have been shown plotted on Drawing Nos. S14-8725-2 and 3. Based on the depths in which groundwater seepage and sloughing was encountered, the groundwater table appears to be situated approximately 2.2 metres below existing ground surface. Higher groundwater conditions could be encountered, particularly during or after periods of precipitation or spring thaw.

3.3 Cobblestones and Boulders

Cobblestones and/or boulders were not encountered within the depths explored during test drilling.

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 and organic content.

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. S14-8725-2 and 3.

The results of the grain size distribution analysis have been enclosed in Appendix D.

5.0 DESIGN RECOMMENDATIONS

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

5.1 Design Considerations

It is understood that the proposed Structures will consist of proposed Gasoline and Diesel Above Ground Storage Tanks (ASTs) and a Propane Fill Station. It is understood that the proposed Gasoline, Diesel and Propane Tanks will be about 10,000 to 11,000 L in volume.

The general subgrade soil conditions consisted of granular fill overlying sand followed by clay. A layer of clay was encountered in Test Hole No. 14-1 at approximately 0.2 metres below grade extending to a depth of about 1.3 metres. The subgrade soils are frost susceptible and the average depth of frost penetration for the Prince Albert area is approximately 2.0 metres for heated buildings and 2.5 metres for unheated structures.

Extensive groundwater seepage and sloughing conditions were encountered within the sand stratum during test drilling. Based on the depths at which groundwater seepage and sloughing was encountered, the groundwater table appears to be situated approximately 2.2 metres below existing ground surface. Higher groundwater levels could be encountered, particularly during and/or following periods of precipitation or spring thaw.

A perimeter edge thickened slab could perform satisfactorily as a foundation system for this site, provided some differential foundation movements are considered acceptable. Potential differential movements would be attributed to the variable soil types at the site and frost action. The clay has the potential to undergo volume changes with fluctuations in moisture content.

In order to minimize any potential movements that may develop, over-excavation and replacement of some of the clay is recommended to improve the load carrying characteristics of the subgrade soils and provide a uniform subgrade condition. A minimum of 600 mm of granular base course fill is recommended beneath the perimeter thickened edge slab.

If potential differential foundation movements are not considered tolerable, a deep foundation system consisting of driven, timber piles or helical screw piles could be considered and should perform satisfactorily at this site.

Drilled, cast-in-place concrete piles were considered but are not recommended due to the extensive groundwater seepage and sloughing conditions expected within the sand stratum.

It should be noted that soil subsidence has historically been observed near the proposed AST pad locations. As such, a deep foundation may be a more viable option for the site.

Recommendations have been prepared for site preparation; thickened edge raft; driven, timber piles; helical screw piles; factor of safety/reduction factors; grade-supported floor slabs; and, foundation concrete.

5.2 Site Preparation

All loose fill, construction debris and deleterious materials should be removed from the construction area. Staining and root intrusion from 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 unsuitable soil which should be removed in preparation of the site for construction. See Appendix B for further information with respect to topsoil composition and soil structure.

The surface of the subgrade should be leveled 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 fill. The fill should be placed in thin lifts (maximum 150 mm loose) and compacted as noted above. 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 the proposed Structures.

5.3 Perimeter Thickened Edge Raft Foundation

The following minimum recommendations should be incorporated into the design of a perimeter edge thickened raft foundation supported on structural granular fill.

1. Over-excavate the clay soils to provide a minimum of 600 mm of granular base course fill beneath the slab. Level and compact the uppermost 150 mm of the subgrade surface to 96 percent of standard Proctor density at optimum moisture content. Do not allow the subgrade surface to dry out. The fill should be placed as soon as possible to minimize drying of the clay soils.

2. Place woven geotextile (Geotex 2x2 HF) (by hand) over the levelled subgrade surface. Overlap adjoining pieces of geotextile a minimum of 600 mm. Over the geotextile, place and level a minimum of 200 mm of sub-base (25 mm maximum size), in a single lift, ensuring equipment is working on top of the lift and not in contact with the geotextile. Compact the granular fill material to 96 percent of standard Proctor density using static compaction equipment (no vibratory equipment).
3. The 600 mm layer of granular fill should be placed and compacted in thin lifts (150 mm loose, maximum) to 100 percent of standard Proctor density at optimum moisture content. The granular fill should extend laterally away from the edge of the raft a distance equal to the depth of the fill.
4. The raft foundation, constructed in accordance with the above recommendations, may be designed to exert an ultimate soil bearing pressure of 275 kPa (ultimate Limit State – ULS). The serviceability Limit State (SLS) bearing pressure equivalent to 25mm of raft settlement is equal to 100 kPa.
5. Continuous quality control inspection should be provided during fill placement.
6. The raft foundation should not be constructed on desiccated or frozen subgrade soil.
7. 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. Placing two layers or using a heavier gauge of poly should be considered to minimize damage to the barrier system. A cushioning layer of bedding sand above and below the vapour barrier is recommended.
8. Frost should not be allowed to penetrate beneath the raft foundation prior to or during construction.

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

To minimize the amount of differential movement associated with potential frost heaving, it is recommended that rigid polystyrene insulation be placed around the perimeter of the raft and should extend under the entire foundation area. The insulation should be at least 50 mm in thickness and should extend out a minimum of 1.2 metres from the edge of the slab foundation. The insulation should be placed a minimum of 300 mm below finished grade and should be sloped away from the structure.

5.4 Driven, Treated Timber Piles

Driven, treated timber piles may be designed on the basis of skin friction only. The ultimate skin friction bearing pressures for driven, treated timber piles are as follows:

TABLE I. SKIN FRICTION BEARING PRESSURES (TIMBER PILES)

Zone (metres)	Ultimate Skin Friction Bearing Pressure (kPa)
0 to 2	0
2 to 5	70
Below 5	50

Notes:

1. For drop hammers, a minimum drop hammer mass of twice the mass of the pile, but not exceeding five times the mass of the pile, is recommended.
2. A pre-bore diameter of at least the pile diameter plus 50 mm should be used through the depth of fill and/or frost penetration. Where piles are pre-bored and subject to lateral loading, a pre-bore annulus that is smaller than the pile diameter (i.e., 90%) is recommended to ensure full contact between the pile shaft and surrounding soil.

3. To minimize the potential for frost jacking, driven, treated timber piles should have a minimum embedment length of 6 metres. If the termination criteria is achieved at a depth which is significantly shallower than the design depth, then the pile capacity should be reviewed. Pre-boring may be required if the termination criteria is achieved prematurely.
4. A minimum centre-to-centre spacing of not less than three pile diameters is recommended.
5. Although not anticipated, timber piles should not be subject to hard driving. The potential problems as a result of hard driving are splitting of the pile, brooming of the pile toe and bowing or breaking of the pile. Pile banding may be required to minimize potential damage during driving. To reduce the potential for damage, driving must be stopped upon satisfying the following termination criteria.

TABLE II. TERMINATION CRITERIA (TIMBER PILES)

Nominal Pile Size mm/No.	Rated Energy Per Hammer Blow (Joules)*	Termination Criteria Hammer Blows for 25 mm Penetration
250/10	25,000 (18,500 ft - lbs)	2
275/11	27,000 (20,000 ft - lbs)	3
300/12	30,000 (22,000 ft - lbs)	3
355/14	35,000 (26,000 ft - lbs)	4

*1 foot - pound - force = 1.356 Joules

6. The structural capacity of each pile should be confirmed by a structural engineer to ensure that over-stressing of the pile does not occur.
7. A representative of the Geotechnical Consultant should inspect and document the installation of each driven, treated timber pile.

5.5 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 skin friction and end bearing pressures for design of screw piles have been presented below.

TABLE III. SKIN FRICTION BEARING PRESSURES (SCREW PILES)

Zone (metres)	Ultimate Skin Friction Bearing Pressure (kPa)
0 to 2	0
2 to 5	35
Below 5	25

TABLE IV. END BEARING PRESSURES (SCREW PILES)

Depth (metres)	Ultimate End Bearing Pressure (kPa)
3 to 5	500
Below 5	375

Notes:

1. The minimum embedment depth of uppermost helix for multi-helix piles should be $\geq 3\text{m}$ 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 metres below grade or $H/D = 5$ (whichever is greater).
3. For determination of skin friction capacity, the effective shaft length may be taken as the depth of embedment of the pile shaft (to the top of the uppermost helix) minus the diameter of the uppermost helix.
4. A minimum centre-to-centre pile spacing of $2.5B$, where B =helix diameter, is recommended.
5. The helical plate shall be normal to the central shaft (within 3 degrees) over its entire length. Multiple helixes (if applicable) should be spaced at increments of the helix pitch to ensure that all helixes travel the same path during installation.
6. Continuous monitoring of the installation torque should be undertaken during installation to determine whether the screw pile has been damaged during installation and to monitor the consistency of the subsurface soils.
7. Screw piles should be designed on the basis of conventional static analysis using the bearing pressures provided in Tables III and IV. Installation torque should be used for monitoring purposes only and not to determine pile capacity.
8. A representative of the Geotechnical Consultant should inspect and document the installation of each screw pile on a continuous basis.

5.6 Factor of Safety/Resistance Factors

When using traditional Working (allowable) Stress Design (WSD) to design the foundations, an appropriate Factor of Safety must be applied to the ultimate bearing pressures presented in this report. PMEL typically recommends a Factor of Safety of 2.5 for compressive loading and 3.5 to 4 for tensile loading. The actual Factor of Safety should be based on the governing design requirements/codes.

As with WSD, an appropriate reduction must be applied to the ultimate bearing pressures (otherwise known as Ultimate Limit State, ULS) when designing the foundations on the basis of Limit States Design (LSD). This is accomplished in the form of using resistance factors (Φ). As per the National Building Code of Canada - NBCC (2010), the following resistance factors are considered appropriate for the design of:

- Deep foundations:
 - Compressive Resistance, $\Phi = 0.4$
 - Tensile Resistance, $\Phi = 0.3$

For both WSD and 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 raft foundation at this site, the provided SLS bearing capacity is based on a settlement of 25 mm. If a lesser settlement is required for the raft foundation, PMEL should re-evaluate the recommended SLS bearing capacity.

Provided the foundation is designed using the appropriate factors of safety or resistance factors presented above, the amount of settlement of a deep foundation at the design load will be small and within tolerable limits (typically less than 10 mm). Hence, settlement typically does not govern in the majority of cases of deep foundation design.

5.7 Grade-Supported Floor Slabs

If a pile and grade-beam foundation system is implemented, the recommendations provided in Section 5.3 – Perimeter Thickened Edge Raft, should be followed for a conventional, lightly loaded grade supported floor slab.

5.8 Foundation Concrete

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

TABLE V. WATER SOLUBLE SULPHATE TEST RESULTS

Test Hole No.	Depth (m)	Soil Type	Water Soluble Sulphate (%)	Class of Exposure	Degree of Sulphate Exposure
14-1	1.0	Clay	0.01	--	Negligible
14-2	7.5	Clay	0.04	--	Negligible

An examination of Table V revealed that the measured sulphate contents varied from 0.01 to 0.04 percent, which is considered negligible to severe in terms of potential degree of sulphate attack. Based on the test results General use cement (CSA Designation GU) may be used for all foundation concrete in contact with the soil. However, water-soluble sulphate salts (gypsum crystals) are known to exist in the geologic deposits in this region. PMEL recommends that sulphate resistant cement be used for all concrete in contact with the subgrade soil. 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.

6.0 LIMITATIONS

The presentation of the summary of the field drill logs and design recommendations has been completed as authorized. Two, 150 mm diameter test holes were dry drilled using our truck-mounted, continuous flight auger drill rig. 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.

One piezocone penetration test (CPTu) was also conducted during the field investigation. The inferred soil stratigraphy has been shown on the attached CPTu plot.

Variations in the subsurface conditions from that shown on the drill logs at locations other than the exact Test Hole 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.

No detectable evidence (i.e., odor or visual) 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 WSP Canada Inc. and their agents for specific application to the proposed Fuel Tank Pad and Fill Station to be constructed within the Saskatchewan Penitentiary in Prince Albert, Saskatchewan. It has been prepared in accordance with generally accepted geotechnical engineering practices and no other warranty, express or implied, is made.

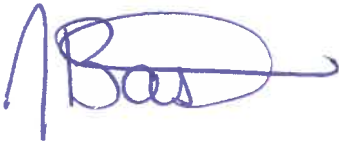
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The acceptance of responsibility for the design/construction recommendations presented in this report is contingent on adequate and/or full time inspection (as required, based on site conditions at the time of construction) by a representative of the 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 this report fulfils your requirements for this project. Should you require additional information, please contact us.

P. MACHIBRODA ENGINEERING LTD.



Jason Bast, Engineer-in-Training

Association of Professional Engineers &
Geoscientists of Saskatchewan

CERTIFICATE OF AUTHORIZATION

P. MACHIBRODA ENGINEERING LTD.

Number 172

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Terry Werbovetski, P. Eng.

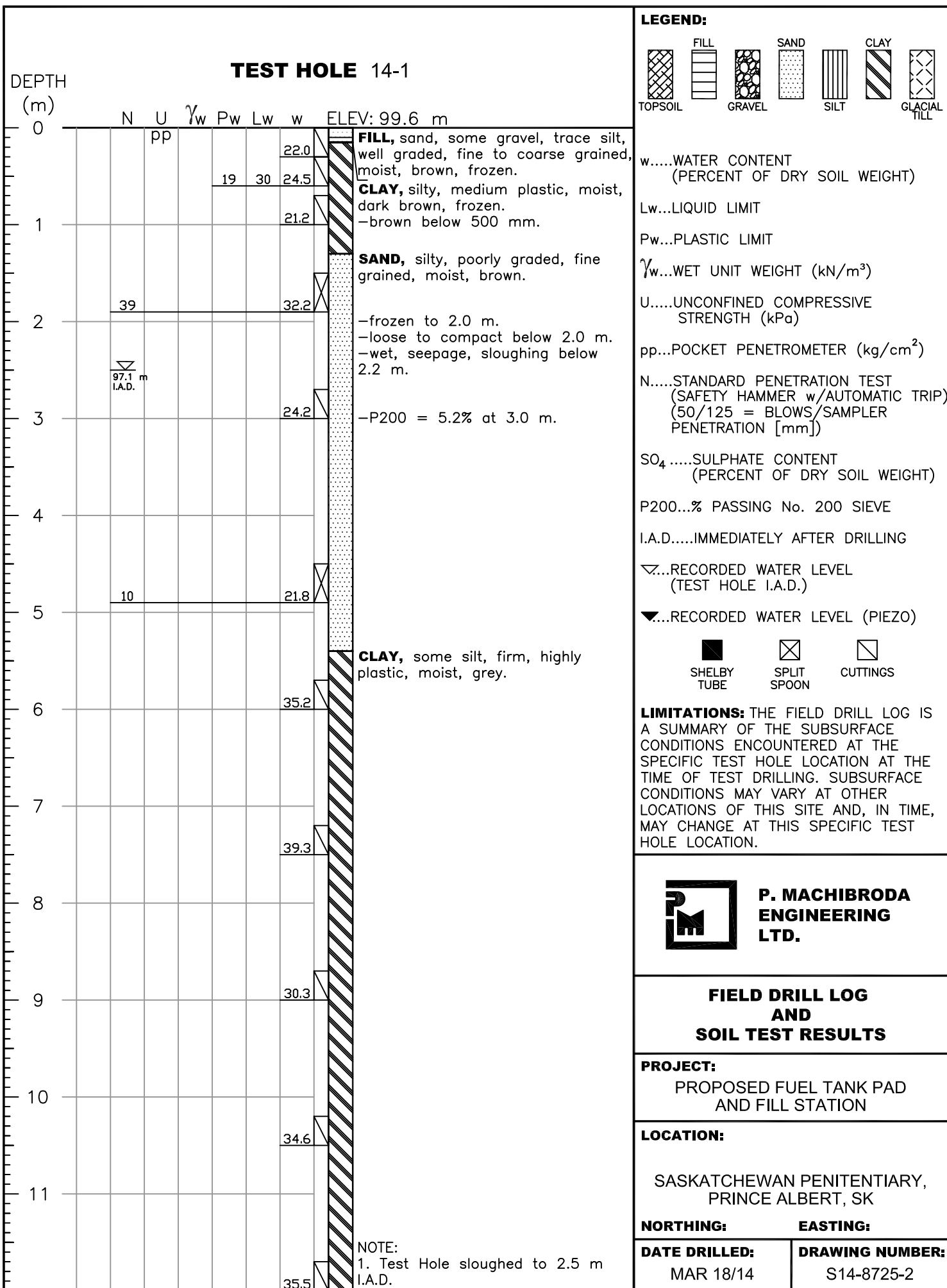
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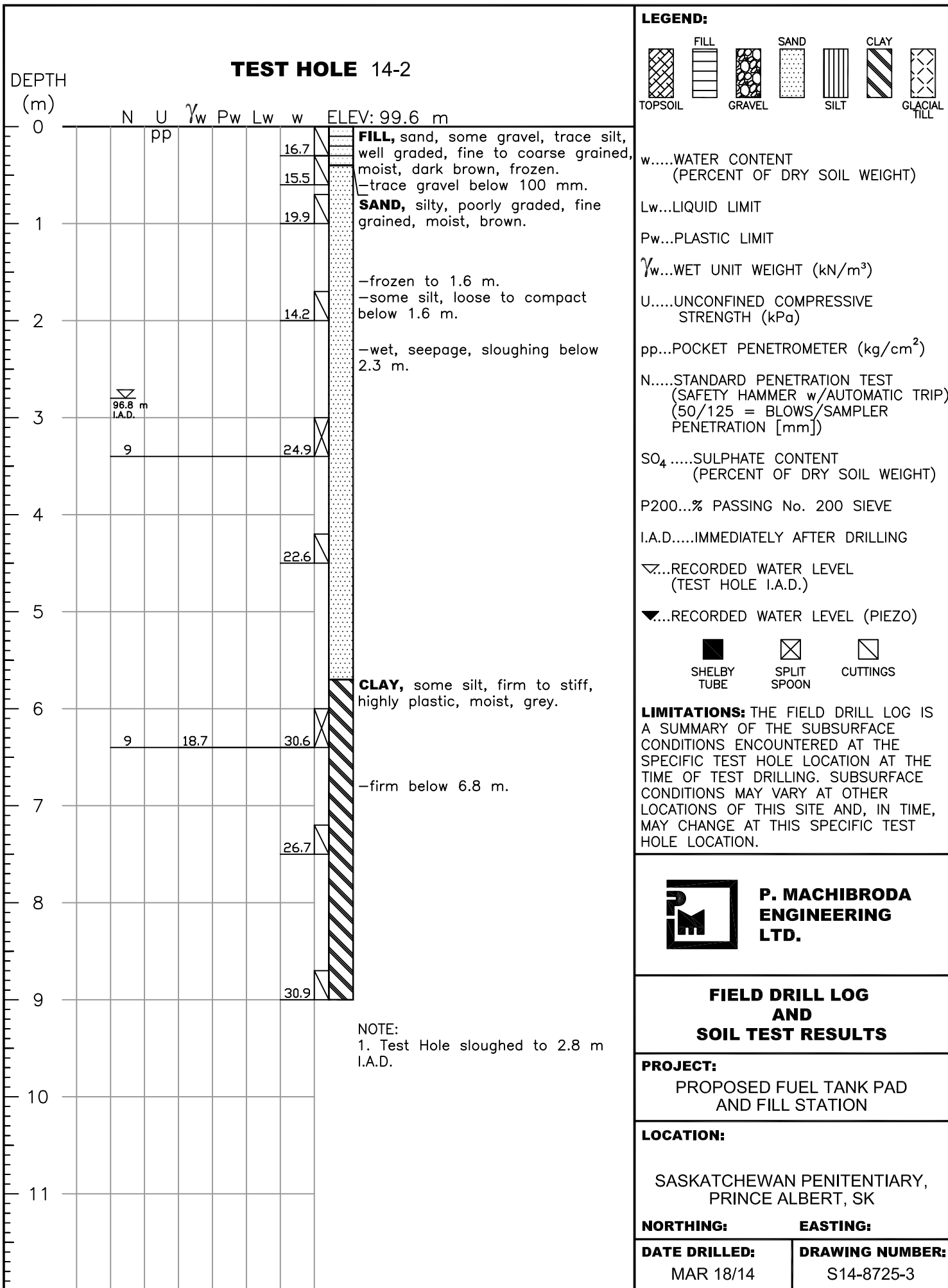




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DRAWINGS





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.

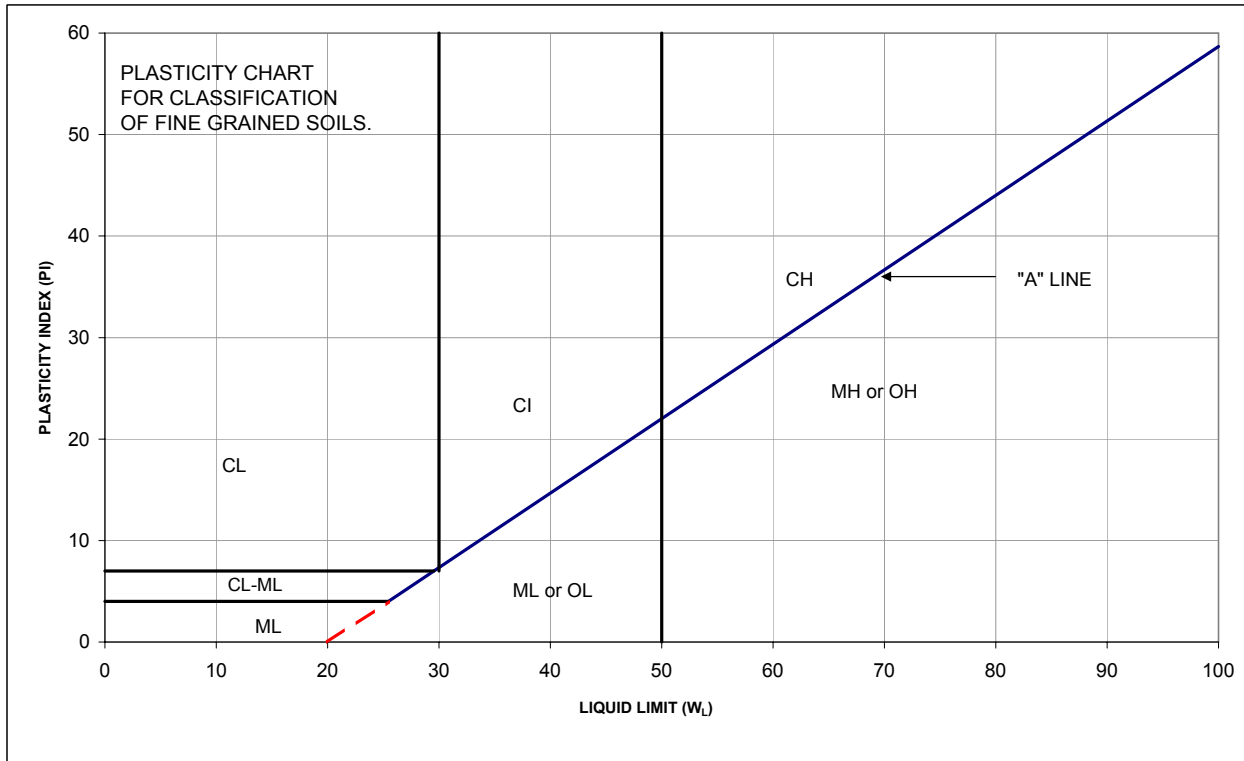
Organic Soils: Readily identified by colour, odour, spongy feel and frequently by fibrous texture.

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.

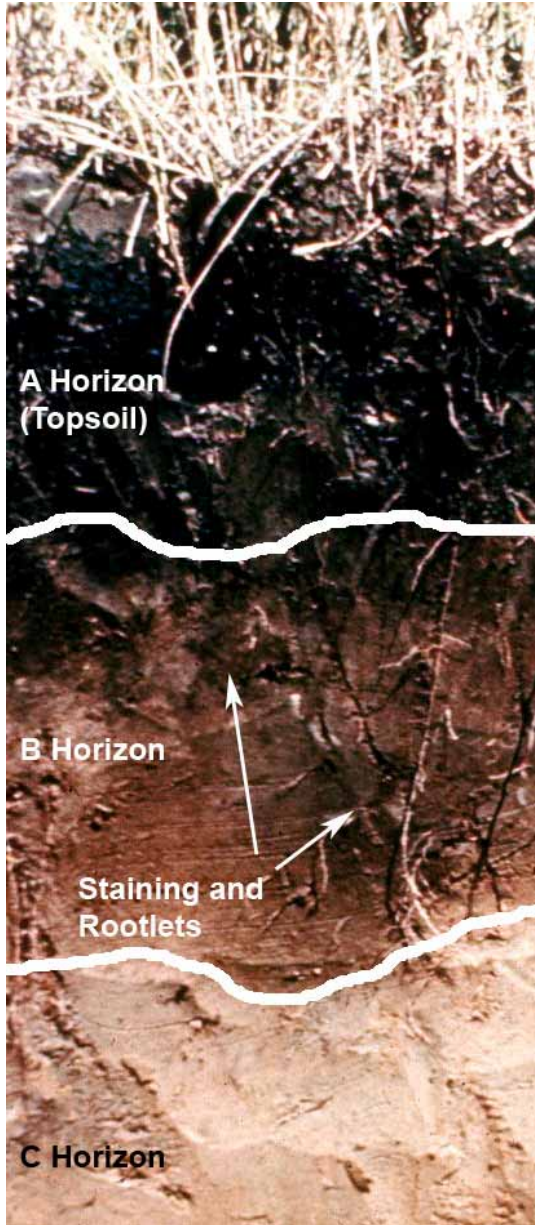
SOIL CLASSIFICATION SYSTEM (MODIFIED U.S.C.)

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	$C_u = \frac{D_{60}}{D_{10}} > 4$ $C_c = \frac{(D_{30})^2}{D_{60} \times D_{10}} = 1 \text{ to } 3$
			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	$C_u = \frac{D_{60}}{D_{10}} > 6$ $C_c = \frac{(D_{30})^2}{D_{60} \times D_{10}} = 1 \text{ to } 3$
			SP	POORLY-GRADED SANDS OR GRAVELLY SANDS <5% FINES	NOT MEETING ALL GRADATION REQUIREMENTS FOR SW
		DIRTY SANDS	SM	SILTY SANDS, SAND-SILT MIXTURES >12% FINES	ATTERBERG LIMITS BELOW "A" LINE OR PI < 4
			SC	CLAYEY SANDS, SAND-CLAY MIXTURES >12% FINES	ATTERBERG LIMITS ABOVE "A" LINE WITH PI >7
FINE-GRAINED SOILS (MORE THAN HALF BY WEIGHT PASSING NO. 200 SIEVE SIZE)	SILTS Below "A" line on plasticity chart; negligible organic content		ML	INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTY SANDS OF SLIGHT PLASTICITY	$W_L < 50$
			MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS, FINE SANDY OR SILTY SOILS	$W_L > 50$
	CLAYS Above "A" line on plasticity chart; negligible organic content		CL	INORGANIC CLAYS OF LOW PLASTICITY, GRAVELLY, SANDY, OR SILTY CLAYS, LEAN CLAYS	$W_L < 30$
			CI	INORGANIC CLAYS OF MEDIUM PLASTICITY, SILTY CLAYS	$W_L > 30 < 50$
			CH	INORGANIC CLAYS OF HIGH PLASTICITY, FAT CLAYS	$W_L > 50$
	ORGANIC SILTS & ORGANIC CLAYS Below "A" line on plasticity chart		OL	ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY	$W_L < 50$
			OH	ORGANIC CLAYS OF HIGH PLASTICITY	$W_L > 50$



APPENDIX B

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.

APPENDIX C

CPTu Plot

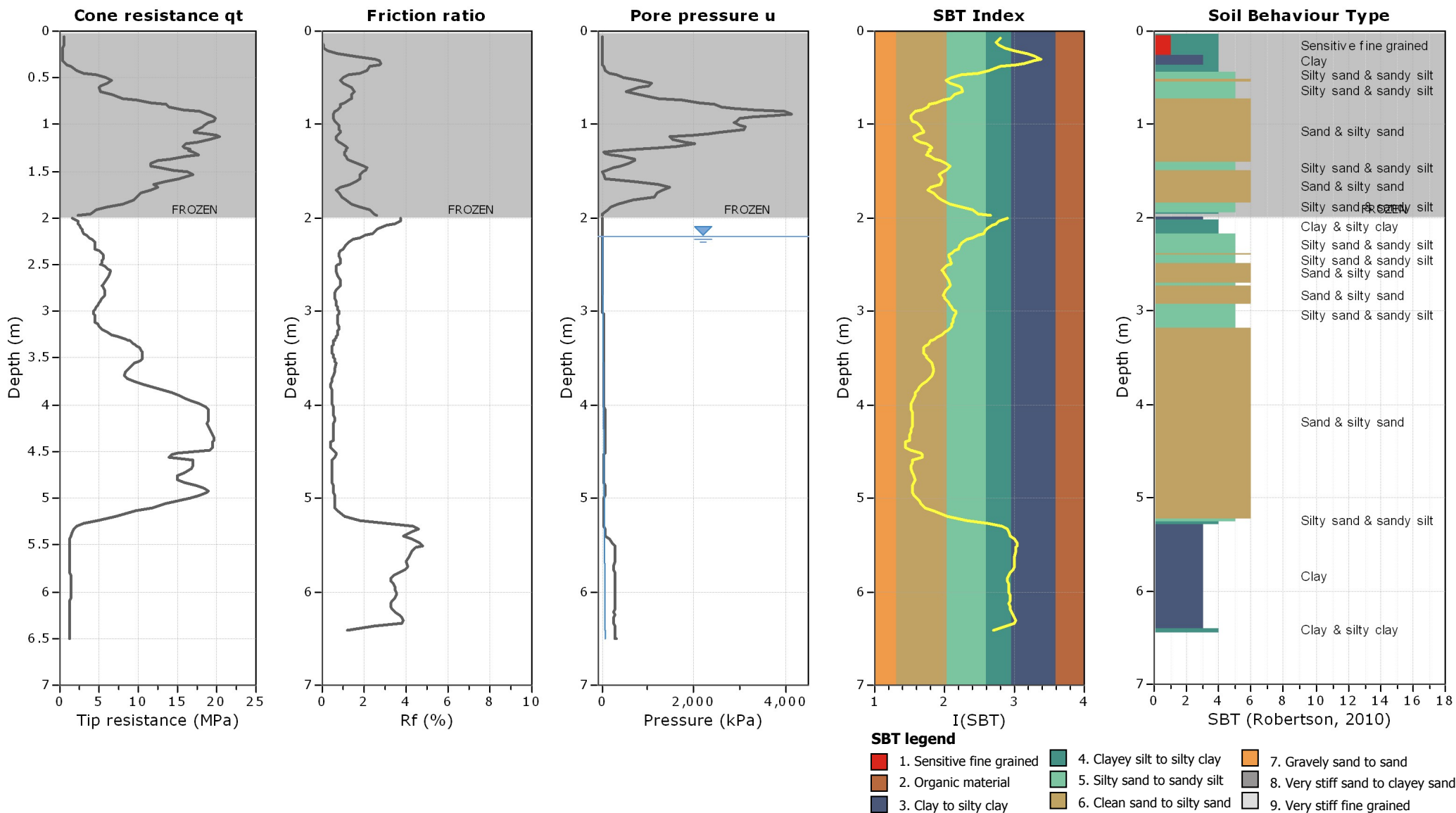


P. Machibroda Engineering Ltd.
806 - 48th Street East, S7K 3Y4
Saskatoon, Saskatchewan
www.machibroda.com

Project: Proposed Fuel Tank Pad & Fill Station
Location: Saskatchewan Penitentiary, Prince Albert

CPT: CPTu 14-1

Total depth: 6.50 m, Date: 3/25/2014
Surface Elevation: 0.00 m
Coords: X:0.00, Y:0.00
Cone Type: Vertek 15cm²
Cone Operator: PMEL



APPENDIX D

Laboratory Results

ASTM D422: GRAIN SIZE ANALYSIS OF SOIL

Project: FUEL TANK PAD AND FILL STATION
P.A. PEN, SASKATCHEWAN

Project No.: S14-8725

Date Tested: MARCH 24, 2014

Test Hole No.: 14-2

Sample No.: 15

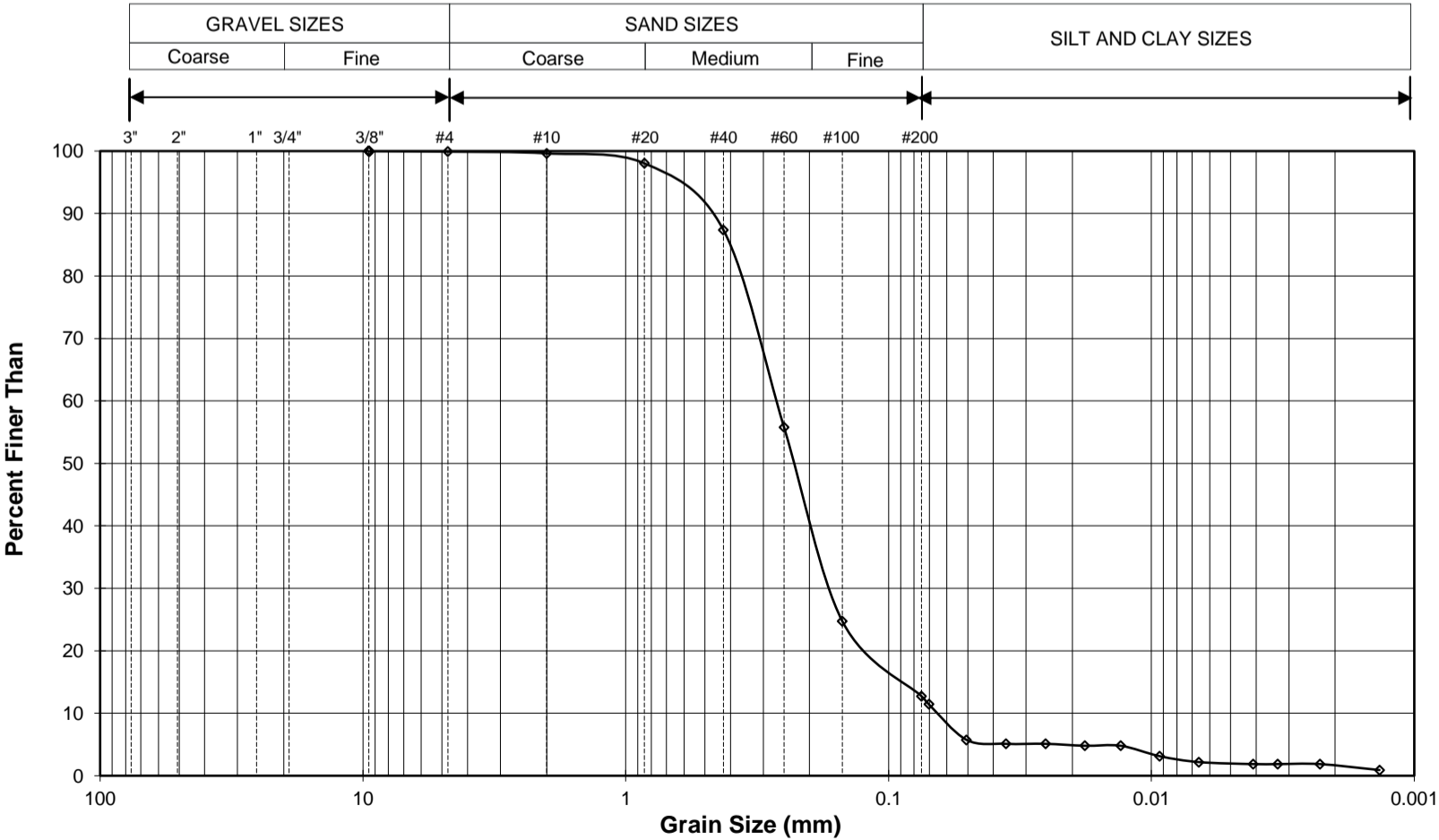
Depth (m): 2.0

Sieve Analysis:	Sieve	Diameter	%	Hydrometer Analysis:	Diameter	%
		mm	Finer		mm	Finer
	1.5"	38.1	100	Dispersing Agent:	0.0703	11.4
	1"	25.4	100	Sodium Hexametaphosphate	0.0506	5.7
	3/4"	19.1	100		0.0358	5.1
	1/2"	12.7	100		0.0253	5.1
	3/8"	9.5	100		0.0179	4.8
	# 4	4.75	100		0.0131	4.8
	# 10	2	100		0.0093	3.1
	# 20	0.85	98		0.0066	2.2
	# 40	0.425	87.4		0.0041	1.9
	#60	0.25	55.8		0.0033	1.9
	# 100	0.15	24.7		0.0023	1.9
	# 200	0.075	12.7		0.0014	0.9

Material Description:

% Gravel Sizes	% Sand Sizes	% Silt Sizes	% Clay Sizes
0	87	11	2

Remarks:



P. MACHIBRODA
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DRAWING NO.

Appendix D - 1