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GEOTECHNICAL INVESTIGATION
PROPOSED ADMINISTRATION BUILDING
WITHIN NW-32-56-6-W5M
LAC STE. ANNE COUNTY
NEAR SANGUDO, ALBERTA

PMEL FILE NO. 9769
DECEMBER 17, 2014

PREPARED FOR:

LAC STE. ANNE COUNTY
C/O MHPM PROJECT MANAGERS INC.
360 MANULIFE PLACE, 10180 101 STREET
EDMONTON, ALBERTA
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1.0 INTRODUCTION

The following report has been prepared on the subsurface soil conditions existing at the site of the proposed Administration Building to be constructed within NW-32-56-6-W5M, in Lac Ste. Anne County, near Sangudo, Alberta.

Authorization to conduct this investigation was provided on November 5, 2014 by Mr. Thomas Chan of MHPM Project Managers Inc. The Terms of Reference for this investigation were presented in P. Machibroda Engineering Ltd. (PMEL) Proposal No. 1028-2985, dated October 29, 2014.

The field test drilling and soil sampling were conducted on November 14 2014. Groundwater monitoring was conducted on November 27, 2014.

2.0 FIELD INVESTIGATION

Seven test holes, located as shown on the Site Plan, Drawing No. 9769-1, 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 3 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.

Standpipe piezometers (slotted, 50 mm diameter PVC pipe) were installed in Test Hole Nos. 14-4 and 14-5 for groundwater monitoring purposes.

3.0 FIELD DRILL LOGS

The field drill logs recorded during test drilling are shown plotted on Drawing Nos. 9769-2 to 8, inclusive.

The coordinates and ground surface elevation at each Test Hole location was surveyed by Hamilton and Olsen Surveys Ltd. On November 26, 2014

3.1 Soil Profile

The general subgrade soil conditions consisted of a thin layer of topsoil overlying highly plastic clay underlain by glacial till, which extended to a depth of at least 12 metres below existing ground surface, the maximum depth explored with our test holes at this site. Organics (peat) were encountered to depths of 0.6 and 1.2 metres below existing grade in Test Hole Nos. 14-4 and 14-6, respectively.

3.2 Groundwater Conditions, Sloughing

The test holes remained open and dry during and immediately following test drilling. A summary of the ground water levels in the installed piezometers have been presented in Table I.

TABLE I. RECORDED GROUNDWATER LEVELS

Test Hole No.	Ground Surface Elevation (m)	Piezometer Rim Elevation (m)	Groundwater Elevation (m)	
			[1]IAD	November 27, 2014
14-4	688.31	689.261	Dry	Dry
14-5	687.97	689.162	Dry	Dry

Note:

1. IAD = Immediately After Drilling.

An examination of Table I revealed that the piezometers were dry on November 27, 2014, indicating the groundwater level was lower than 12 metres below existing grade (i.e., the extent of our test holes).

It should be recognized that the piezometers may not have achieve static equilibrium. Higher or perched groundwater conditions could be encountered, particularly after piezometer equilibrium, and during or following precipitation and/or spring thaw.

3.3 Cobblestones, Boulders and/or Bedrock

The glacial till consisted of a heterogeneous mixture of gravel, sand, silt and clay-sized particles. The glacial till strata also contained sorted deposits of the above particle sizes. In addition to the sorted deposits, a random distribution of larger particle sizes in the cobblestone range (60 to 200 mm) and boulder-sized range (larger than 200 mm) were encountered during test drilling.

It should be recognized that the statistical probability of encountering cobblestones and/or boulders in the seven small diameter Test Holes drilled at this site was low. Intertill deposits of cobblestones, boulders, boulder pavements and isolated deposits of saturated sand or gravel should be anticipated. The frequency of encountering such deposits will increase proportionately with the number of holes drilled or volume of soil excavated.

4.0 LABORATORY ANALYSIS

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

The results of soil classification and index tests conducted on representative samples of soil recovered from this site have been plotted on the drill logs alongside the corresponding depths at which the samples were recovered as shown on Drawing Nos. 9769-2 to 8, inclusive.

The results of the grain size distribution analyses are shown plotted in Appendix B.

5.0 DESIGN RECOMMENDATIONS

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

5.1 Design Considerations

It is understood that the proposed Administration Building will be approximately 2,500 square metres (27,000 square feet) in plan area, with an adjacent parking lot that will accommodate 60 vehicles. It is also understood that the building may have a 2 to 3 metre deep basement, approximately 372 square metres (4,000 square feet) in plan area.

The general soil profile consisted of top soil or organics (peat) overlying clay followed by glacial till. The subgrade soils are frost susceptible and the average depth of frost penetration for the Sangudo, Alberta area is approximately 2 metres (1.5 metres for unheated structures). The installed piezometers were dry on November 27, 2014, indicating that the groundwater level was lower than 12 metres below existing grade. Higher or perched groundwater conditions may be encountered during or following precipitation events or snowmelt.

A deep foundation system consisting of drilled, cast-in-place concrete piles and/or belled cassions should perform satisfactorily at this site. Alternatively, driven, steel pipe piles or helical screw piles could also perform satisfactorily at this site.

The near surface subgrade soil conditions consisted of highly plastic clay. Shallow foundations and grade supported concrete floor slabs based on highly plastic soils would be subjected to differential movements associated with seasonal variations in the natural moisture content. As such, a shallow footing foundation is not recommended at this site. If some differential movement cannot be tolerated for grade-support concrete floor slabs, a structural floor slab should be constructed.

Recommendations have been prepared for site preparation; drilled, cast-in-place concrete piles/belled caissons; driven steel pipe piles; helical screw piles; limit states resistance factors and serviceability; floor slabs; foundation walls and drainage; lateral earth pressures; excavations and de-watering; grade beams; foundation concrete; site classification for seismic site response and asphalt concrete pavement .

5.2 Site Preparation

Up to 1.2 metres of organics (peat) was encountered within the building footprint. All organics, topsoil, unsuitable fill material and deleterious materials should be removed from the building footprint and parking areas. 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 surficial soils which should be removed in preparation of the site for construction. See Appendix C for further information in regards to topsoil composition and soil structure.

The design subgrade elevation should be levelled and compacted to the following minimum density requirements.

Building Area	-	98 percent of standard Proctor density at optimum moisture content;
Roadway and Parking 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 fine grained soils. The fill should be placed in this lifts (maximum 150 mm loose) and compacted to 98 percent of standard Proctor density at optimum moisture content. The 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.

5.3 Drilled, Cast-In-Place Concrete Piles and/or Belled Caissons

Drilled, straight shaft, cast-in-place, reinforced concrete piles may be designed on the basis of skin friction only. Belled caissons may be designed on the basis of skin friction and end bearing capacity.

The ultimate skin friction bearing pressures of the undisturbed soil are as follows:

TABLE II. SKIN FRICTION BEARING PRESSURES (DRILLED PILES)

Zone (metres)	Ultimate Skin Friction Bearing Pressure (kPa)
0 to 2	0
2 to 12	45
Below 12	70

Notes:

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 conditions are encountered to maintain the pile holes open for placing of the reinforcing steel and 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.

TABLE III. END BEARING PRESSURE (BELLED CAISSONS)

Depth (metres)	Ultimate End Bearing Pressure (kPa)
Below 12	650

Notes:

1. End bearing caissons designed on the basis of 650 kPa should bear on undisturbed, naturally deposited stiff glacial till.

2. For determination of skin friction capacity, the effective shaft length for belled caissons may be taken as the depth of embedment of the straight sided portion of the pile shaft, minus a length equal to the pile diameter (i.e., the bottom-most portion of the pile shaft is neglected to account for interaction with the bell).

3. End bearing caissons should be inspected to confirm the removal of loose, disturbed soil prior to placing concrete and steel.

4. Caisson shafts should be reinforced.

5. Concrete should be placed as soon as practical after cleaning the bell.

6. 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. As casing is extracted, concrete in casing must have adequate head to displace all water in the annular space.

7. End bearing caissons may be belled at the base to a maximum of three times the shaft diameter.

8. The height of the bell should be designed to provide adequate concrete to distribute the unit stresses into the concrete without over-stressing the outer, non-reinforced concrete within the bell.

9. Full time inspection by a representative of the Geotechnical Consultant, employed directly by the Owner, is required to confirm pile bearing pressures and to document the installation of each end bearing caisson.

5.4 Driven, Open-End Steel Pipe Piles

Driven, open-end steel pipe piles may be designed on the basis of skin friction and end bearing capacity. The ultimate skin friction and end bearing pressures for driven, open-end steel pipe piles have been presented in Tables IV and V.

TABLE IV. SKIN FRICTION BEARING PRESSURES (DRIVEN, PIPE PILES)

Zone (metres)	Ultimate Skin Friction Bearing Pressure (kPa)
0 to 2	0
2 to 12	40
Below 12	70

TABLE V. END BEARING PRESSURES (DRIVEN, PIPE PILES)

Zone (metres)	Ultimate End Bearing Pressure (kPa)
Below 12	900

Notes:

1. A minimum pipe wall thickness of 8 mm is recommended.

2. To minimize the potential for frost jacking, driven, open-end steel pipe piles should have a minimum embedment length of 6 metres.

3. A minimum pile spacing of three times the shaft diameter is recommended.

4. If a high driving resistance should be encountered at a significantly shallower depth than the calculated minimum length required for the design load, then it may be necessary to drill out the soil from within the open-end pipe pile to achieve the minimum pile design embedment length. If the pile terminates at a depth significantly less than the design length, the pile capacity should be reviewed by the Geotechnical Consultant.

5. Piles should not be subject to additional driving if the following termination criteria has been met.

TABLE VI. TERMINATION CRITERIA (DRIVEN, PIPE PILES)

Nominal Pile Diameter (mm)	Minimum Wall Thickness (mm)	**Energy per Hammer Blow (kilojoules)*	Termination Criteria (Hammer Blows for 25 mm Penetration)
195	8	25	4
219	8	30	4
250	8	30	4
300	9.5	30	6
400	9.5	35	6
500	12.5	40	8
600	12.5	40	8

*Note: 1 foot-pound-force = 1.356 Joules

** Maximum driving energy 600kJ/cm^2 of end steel area.

6. 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, the pre-bore annulus should be restricted to 90% of the pile diameter to ensure full contact between the pile shaft and surrounding soil.

7. The installation of each pile and the elevation monitoring of each pile within nine pile diameters which could be affected by the installation of adjacent piles should be documented during construction by a representative of the Geotechnical Consultant. Each pile should be inspected for damage as a result of the driving operations and for the accumulation of water.

8. After inspection of the pile is complete and upon removal of any accumulated water in the pipe, the pipe should be filled with wet cast (minimum 100 mm slump) 25 MPa concrete. If it is not possible to remove the water from the pipe pile, then the concrete should be placed using a tremie, or an anti-washout admixture could be added to the wet mix concrete to minimize segregation of the concrete in the water.

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. Due to the dense nature of the sand and potential for shallow pile termination, piles with multiple helixes are not recommended at this site.

Pile capacity is derived from shearing resistance along the pile shaft (i.e., skin friction) as well as end bearing capacity of the helix.

The ultimate skin friction and end bearing pressures for the design of screw piles have been presented in Tables VII and VIII.

TABLE VII. SKIN FRICTION BEARING PRESSURES (SCREW PILES)

Zone (metres)	Ultimate Skin Friction Bearing Pressure (kPa)
0 to 2	0
2 to 12	25
Below 12	50

TABLE VIII. END BEARING PRESSURES (SCREW PILES)

Depth (metres below existing grade)	Ultimate End Bearing Pressure (kPa)
	Compressive
6 to 12	550
Below 12*	900

* Screw piles must achieve a minimum installation torque to confirm penetration into stiff glacial till.

Notes:

1. Single helix screw piles should extend to a minimum depth of 5 metres below grade or H/D = 5 (whichever is greater), where H = depth to the helix, D = helix diameter.

2. For determination of skin friction capacity, the effective shaft length (L_{eff}) may be taken as the depth of embedment of the pile shaft (to the top of the helix, H) minus a length equal to the diameter of the helix (D), $L_{eff} = H - D - 2$.

3. End bearing capacity may be calculated utilizing the overall cross-sectional area of the 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.

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 provided bearing pressures presented in Tables VII and VIII and appropriate resistance factors as presented in Section 5.6 Installation torque should be used for monitoring purposes only and not for determination of 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 Limit States Resistance Factors and Serviceability

As per the National Building Code of Canada - NBCC (2010), the following resistance factors (Φ) may be applied to the ultimate bearing pressures presented in the previous sections of the report:

- Deep Foundation:
  - Compressive Resistance, Φ = 0.4
  - Uplift Resistance, Φ = 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.

For pile foundations, provided the foundation is designed using the appropriate factors of safety or resistance factors presented above, the amount of settlement of a deep pile foundation at the design load will be small and within tolerable limits. Hence, settlement typically does not govern in the majority of cases of deep pile foundation design. The exception to this would be the case where belled caissons are employed to carry the foundation load and a large bell diameter is required. Foundation settlement should be evaluated for this scenario.

5.7 Floor Slabs

5.7.1 Grade-Support Floor Slabs

The near surface subgrade soils consisted of highly plastic clay. Grade-supported floor slabs bearing on highly plastic soils have the potential to undergo some differential movements related to seasonal volume changes in highly plastic clay.

Providing a positive drainage alongside the foundation, extending downspouts well away from the building and eliminating irrigation alongside the foundation would serve to minimize the potential for increasing soil moisture content adjacent to the foundation and potential swelling.

The subgrade highly plastic clay soils should also not be allowed to dry out during the construction to minimize shrinkage and subsequent swelling upon wetting.

Since the amount of slab movement depends on many factors (i.e., swelling potential of the clay, existing overburden pressure, existing soil moisture regime, availability of free water, etc.) and is difficult to quantify, measures should be taken to accommodate potential swelling by constructing structural elements such as partition walls, staircases, grade beams, columns, etc. independent of the slab. If some differential movement cannot be tolerated, a structural floor slab should be constructed

The following minimum provisions should be incorporated into the design of heated, grade-supported, cast-in-place, reinforced concrete slabs.

1. Prepare the site in accordance with Section 5.2, Site Preparation. Provide a minimum of 150 mm of crushed granular base course material between the subgrade soil and underside of the floor slab.

2. Level and compact the upper 150 mm of the subgrade soil to a minimum of 98 percent of standard Proctor density at optimum plus 2 percent moisture content. Cover the prepared subgrade with approved fill material as soon as possible to minimize the potential for drying of the subgrade soils.

3. Excavate soft subgrade areas and replace with suitable, non-expansive fill, placed and compacted to 98 percent of standard Proctor density at optimum moisture content.

4. If required, place subgrade fill (i.e., granular fill or non-expansive, fine grained soil) thin lifts (maximum 150 mm loose) and compacted to 96 percent of standard Proctor density at optimum moisture content.

5. The granular base course should be placed in thin lifts (150 mm loose, maximum) and compacted to 98 percent of standard Proctor density.

6. The modulus of subgrade reaction for the prepared subgrade and compacted granular fill will be in the order of 35 MN/m^3.

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

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

9. Provide positive site drainage away from the proposed structures. Extend the downspouts a minimum of 3 metres away from the foundation.

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

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

5.7.2 Structural Floor Slabs

A structural floor can be constructed on compressible void filler. A continuous sheet of polyethylene plastic should be placed between the compressible void filler and the concrete floor. Due the highly plastic nature of the clay soils on site, the structural floor slab should be designed at a minimum to resist the pressure it takes to compress the void filler. The compressible void filler should be installed in accordance with the manufacturer's specifications.

5.7.3 Basement Floor Slabs

Basement floor slabs bearing on highly plastic clay have the potential to undergo some differential movements associated with moisture fluctuations in the soil profile.

Since the amount of slab movement depends on many factors (i.e. swelling potential of the clay, existing overburden pressure, soil moisture regime, availability of free water, etc.) and is difficult to quantify, measures can be taken to accommodate potential swelling by constructing structural elements such as partition walls, staircases, grade beams, columns, etc. independent of the slab.

A minimum thickness of 300 mm of compacted granular fill is recommended beneath the slab. To facilitate drainage, the fill should consist of well graded, clean drainage aggregate as specified in Table IX. Fill placement and compaction should be undertaken in accordance with the recommendations presented above. A polyethylene vapour barrier is recommended beneath the slab to separate the slab from the underlying fill.

TABLE IX. GRADATION REQUIREMENTS – DRAINAGE AGGREGATE

Sieve Designation (mm)	Percent Passing
25.0	100
10.0	60 – 100
5.0	44 – 90
2.0	20 – 80
0.800	0 – 53
0.400	0 – 32
0.160	0 – 10
0.080	0 – 3

A sump pit(s) is recommended below basement floor slabs to collect any free water which may accumulate beneath the floor, and, to collect water from the perimeter drainage system. The surface of the subgrade should be positively graded towards the sump pit(s). The sump pit(s) should be perforated to allow water to drain in from the sub-slab drainage layer.

Any water collected in the sump pit/weeping tile drainage system (i.e., surface runoff water that infiltrates alongside the foundation walls and/or groundwater) should be directed well away from the building (minimum distance of 10 metres is recommended).

The depth below grade and size of the basement area (if one is to be constructed) was not known at the time of our investigation. If the basement level is constructed near the elevation of the groundwater table and/or encompasses a large area, inclusion of weeping tile within the drainage aggregate is recommended to expedite drainage to the sump pit(s).

5.8 Foundation Walls and Drainage

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 15.7 kN/m^3 (100 pcf) if the existing subgrade soils are used for backfill against the wall. For walls backfilled with imported, clean, free-draining granular fill, the lateral earth pressure may be calculated on the basis of an equivalent fluid pressure distribution of 9.4 kN/m^3 (60 pcf). The surcharge loading should be calculated on the basis of actual loads.

The lateral earth pressure loading assumes that the backfill will be placed in thin lifts (maximum 300 mm loose), will be lightly compacted and a peripheral (perforated pipe) drainage system will be installed alongside the foundation walls with the invert elevation set at the base of the wall.

The drainage pipe should be positively drained into a sump pit(s) equipped with an automatic sump pump(s). The perforated drainage pipe should be at least 100 mm in diameter, installed on non-woven geotextile capable of transmitting a flow of not less than 50 litres per second per square metre (ASTM D-4491) placed on the undisturbed foundation soil or free draining sand, as may be required for levelling.

The geotextile should be used to encapsulate at least 300 mm (above invert) of clean, drainage aggregate. In the zone 300 mm above the drainage pipe invert elevation to within 500 mm of ground surface, the fill should consist of clean, free-draining granular material containing less than 5 percent material finer than 0.080 mm. The uppermost 500 mm of backfill should be clay or other low permeability material.

5.9 Lateral Earth Pressures

The presumptive earth pressures presented in Section 5.8 are recommended for use in the design of foundation walls. Alternatively, the following soil parameters may be utilized in the design of walls subject to lateral loading.

TABLE X. EARTH PRESSURE COEFFICIENTS

Soil Type	Effective Angle of Internal Friction	[1]Earth Pressure Coefficients			Unit Weight (kN/m^3)
		Active (K_a)	Passive (K_p)	At- Rest (K_o)	
Clay	20°	0.5	2.0	1.0	18.5

Notes:

1. At-rest earth pressure recommended for design of basement (or other) rigid walls.

For retaining walls, it is recommended to install weep holes spaced at regular intervals at the base of the wall to minimize development of excess water pressure on the back of the wall. The backfill alongside the wall should consist of clean, free, draining granular fill to promote drainage. Highly plastic clay is not recommended for use as backfill material. The backfill should be lightly compacted to minimize development of excess pressure on the wall.

5.10 Excavations and De-Watering

It is anticipated that proposed excavations at this site will be completed with unbraced, sloped side walls. The long term 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 consistency and structure (degree of fracturing, slickensiding, etc.) of the subgrade soils.

Temporary excavations should be designed and excavated in strict compliance with rules and regulations of Alberta Occupational Health and Safety Act (OH & S). The Contractor is solely responsible for protecting excavation by shoring, sloping, benching and/or other means as required to maintain the stability of both the excavation sides and the bottom. The required conformance standards are detailed in Occupational Health and Safety Code 2009, Part 32, Sections 441 to 455, inclusive.

The subsurface conditions at this site have been classified as per OH&S (Section 442) and have been presented in Table XI.

TABLE XI. OH&S SOIL CLASSIFICATION

Material Type	OH&S Soil Classification	PMEL Recommended Slope
Clay/Glacial Till	Likely to crack and crumble	1H:1V

The highly plastic clay and glacial till soils are considered low permeable. Therefore, any groundwater seepage into open excavations should not be significant and readily handled using perimeter drainage trenches directed to sump pits equipped with sump pumps.

5.11 Grade Beams

Grade beams should be reinforced at both top and bottom throughout their entire length. Grade beams should be constructed to allow a minimum of 150 mm of net void space between the underside of the grade beam and subgrade soil.

5.12 Foundation Concrete

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

TABLE XII. WATER SOLUBLE SULPHATE TEST RESULTS

Test Hole No.	Depth (m)	Soil Type	Water Soluble Sulphate (%)	pH	Degree of Sulphate Exposure
14-6	6.0	Clay	0.010	7.63	Negligible
14-7	7.5	Clay	0.090	7.83	Negligible

An examination of Table XII revealed that the measured sulphate contents were 0.010 and 0.090 percent for the two samples tested. Although the potential degree for sulphate attack is negligible based on the results of the testing, it is possible that higher sulphate concentration exist at different depths/locations at this site. Therefore, as a minimum, it is recommended to design the foundation concrete in contact with the subgrade soils on the basis of a Class S-3 Exposure. All concrete at this site should be manufactured in accordance with current CSA standards.

5.13 Site Classification for Seismic Response

Based on the consistency of the subgrade soils encountered at this site and Table 4.1.8.4.A of the 2010 National Building Code, the site classification for seismic site response falls within Class D.

5.14 Asphalt Concrete Pavements

The following minimum recommendations should be incorporated into the design of the asphalt concrete pavement and/or gravelled structures.

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. If wet and/or unworkable areas of weak subgrade are encountered, the soft areas should be excavated and replaced with non-expansive fine grained soils and compacted to 96 percent of standard Proctor density.

3. The need for special measures (i.e., overexcavation, geotextile, geogrid, cement stabilization and/or gravel fill) in soft areas must be subject to review by the Geotechnical Consultant during field construction. Based on the actual conditions encountered at the time of construction, the pavement structure may need to be modified to accommodate the construction equipment and the intended use. To minimize disturbance of the subgrade soil, all necessary excavation should be performed with a backhoe or gradall equipment. The excavating/hauling equipment should not be allowed to travel on the prepared subgrade. The use of light equipment will be required for moisture conditioning, levelling and compaction of the uppermost 150 mm of the subgrade at final design elevation. The granular sub-base should be placed by the end-dump method to avoid heavily loaded traffic on the prepared subgrade.

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

TABLE XIII. THICKNESS DESIGN FOR PAVEMENT STRUCTURES

Pavement Structure	**Heavy Truck Traffic Wheel Loading (5,400 kg) (mm)**	**Medium Truck Traffic Wheel Loading (3,180 kg) (mm)**	**Light Truck/Passenger Vehicle Traffic Wheel Loading (1,830 kg) (mm)**
*Asphalt Concrete	100	75	65
Granular Base	200	150	150
Granular Sub-Base	300	225	150
Prepared Subgrade	(150)	(150)	(150)
Total Thickness	**600**	**450**	**365**

* Use 2:1 equivalency for granular substitution of asphalt concrete.

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

6. All granular fill placed above the subgrade elevation should be placed in thin lifts (150 mm loose) and compacted to a minimum of 98 percent of standard Proctor density. The granular base and sub-base course material should meet the following aggregate gradation requirements presented in Table XIV.

TABLE XIV. AGGREGATE GRADATION REQUIREMENTS

Grain Size (mm)	Percent Passing	
	*Base Course	**Sub-Base Course
80.0	--	100
50.0	--	55-100
25.0	100	38-100
20.0	82-97	
16.0	70-94	32-85
10.0	52-79	
5.0	35-64	20-65
1.25	18-43	
0.630	12-34	
0.315	8-26	6-30
0.160	5-18	
0.080	2-10	2-10
Plasticity Index (%)	0-6	0-8
% Fracture (min.)	60	--

*Alberta Transportation – Designation 2, Class 25
**Alberta Transportation – Designation 6, Class 80

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

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

9. Periodic maintenance such as crack sealing will be required for asphalt concrete pavement. For gravelled structures, periodic grading will be required to maintain the desired riding surface.

6.0 LIMITATIONS

The presentation of the summary of the field drill logs and design recommendations has been completed as authorized. Seven, 150 mm diameter test holes were dry drilled using a 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.

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 of environmentally sensitive materials such as hydrocarbon odour was detected during the actual time of the field test drilling program. If on the basis of any knowledge, other than that formally communicated to us, there is reason to suspect that environmentally sensitive materials may exist, then additional test holes should be drilled and samples recovered for chemical analysis.

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

This report has been prepared for the exclusive use of MHPM Project Managers Inc. and their agents for specific application to the proposed Administration Building to be constructed within NW 32-56-6-W5M, in Lac Ste, Anne County, near Sangudo, Alberta. 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 Parties. Governing Agencies such as municipal, provincial or federal agencies having jurisdiction 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 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 fulfills your requirements for this project. Please contact our office if you should require additional information.

P. MACHIBRODA ENGINEERING LTD.

Dec 17, 14
Farkhideh

Naser Farkhideh, M.Sc., E.I.T.

GRAHAM BAXTER
2014-12-17

Graham Baxter, P.Eng.

NF:GB:zz:clb

PERMIT TO PRACTICE

P. Machibroda Engineering Ltd.

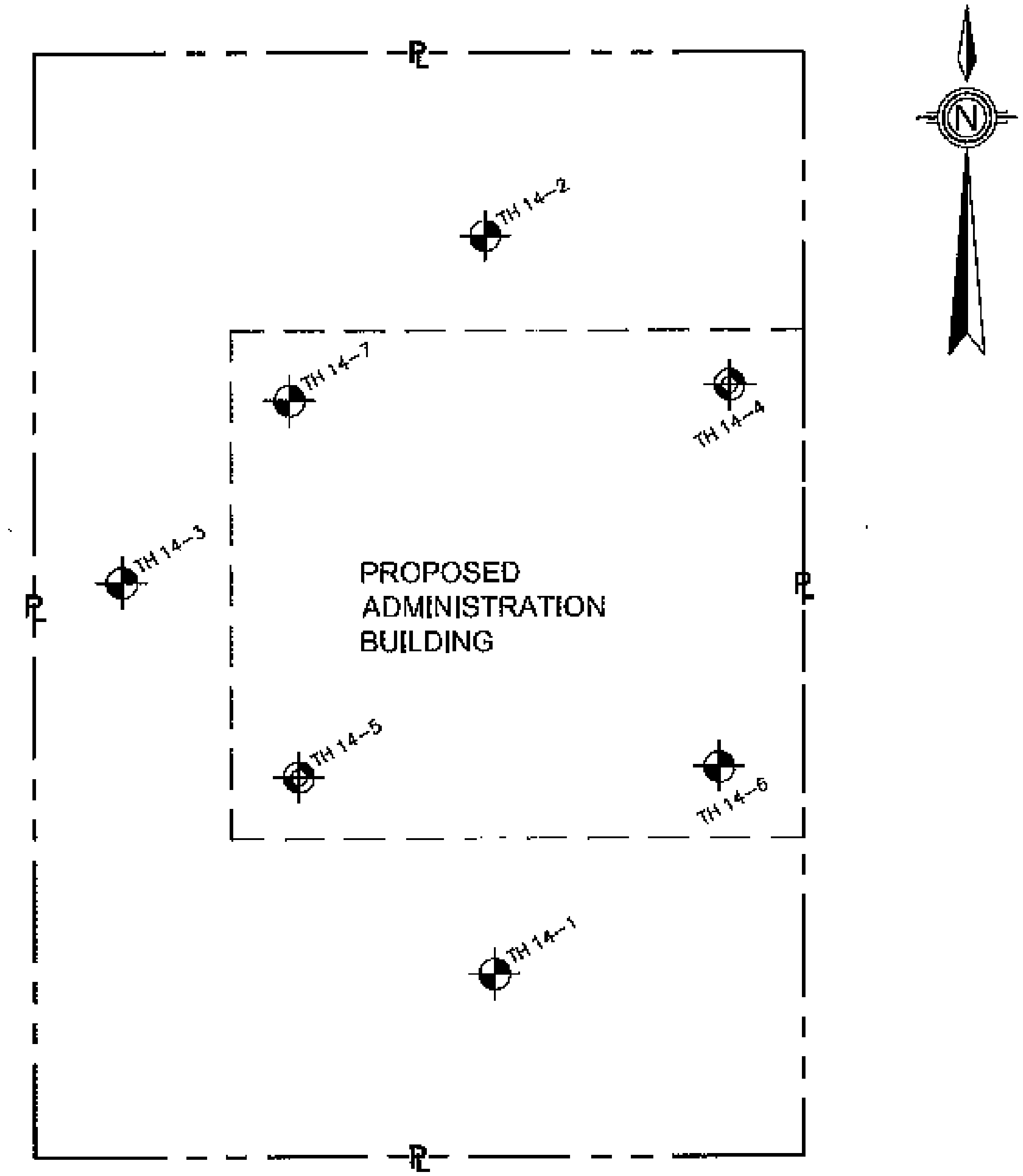
Signature ____________
Date 2014-12-17

PERMIT NUMBER: P-5862

The Association of Professional Engineers and Geoscientists of Alberta

P. MACHIBRODA
ENGINEERING LTD.
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. THIS DRAWING WAS COMPILED FROM SITE PLAN PROVIDED BY MHPM PROJECT LEADER.

LEGEND ⊕ -PMEL TEST HOLE ⊕ -PMEL TEST HOLE (PIEZOMETER INSTALLED) PL -PROPERTY LINE

CONSULTING
GEOENVIRONMENTAL
GEOTECHNICAL
ENGINEERS

P. MACHIBRODA ENGINEERING LTD.

12114A – 163 STREET N.W.
EDMONTON, AB
T5V 1H4

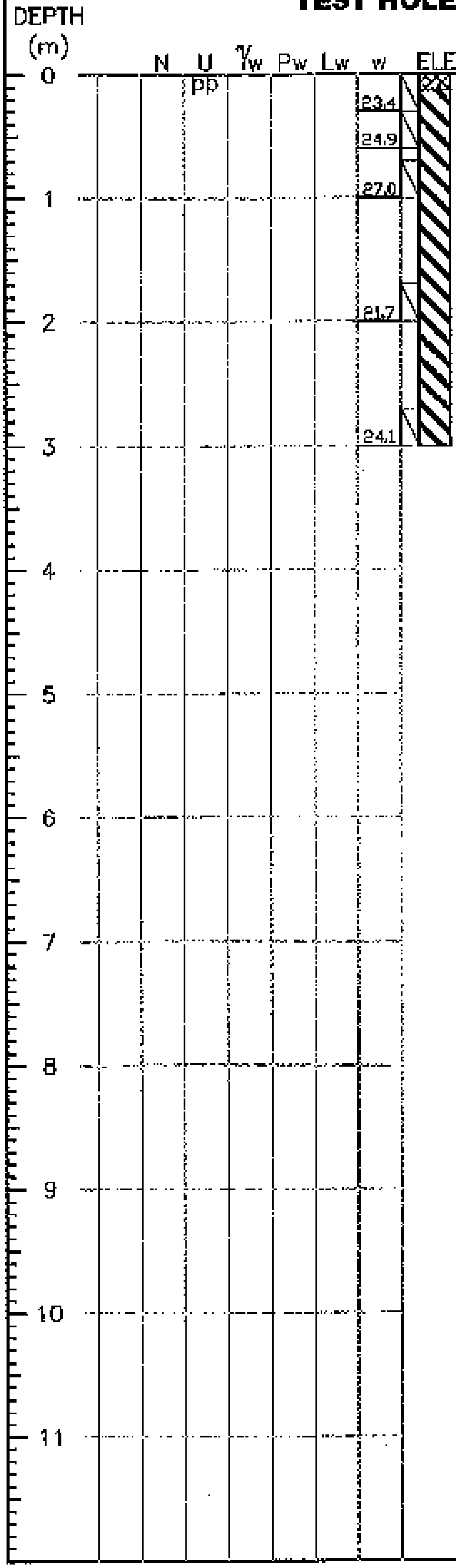
DRAWING TITLE:
SITE PLAN - TEST HOLE LOCATIONS

PROJECT:
PROPOSED ADMINISTRATION BUILDING
WITHIN NW-32-56-6-W5M, LAC STE ANNE COUNTY, NEAR SANGUDO, AB

APPROVED BY: JB/ML	DRAWN BY: LD/SD	DRAWING NUMBER: 9769-1
DATE: NOVEMBER, 2014	SCALE: NOT TO SCALE	

TEST HOLE 14-1

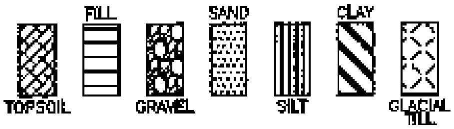
ELEV: 687.7



TOPSOIL, black, grass rootlets.
CLAY, some silt, stiff, highly plastic, moist, grey, oxide stained.

NOTE:
1. Test Hole open to 3.0 m and dry I.A.D.

LEGEND:



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 ADMINISTRATION BUILDING

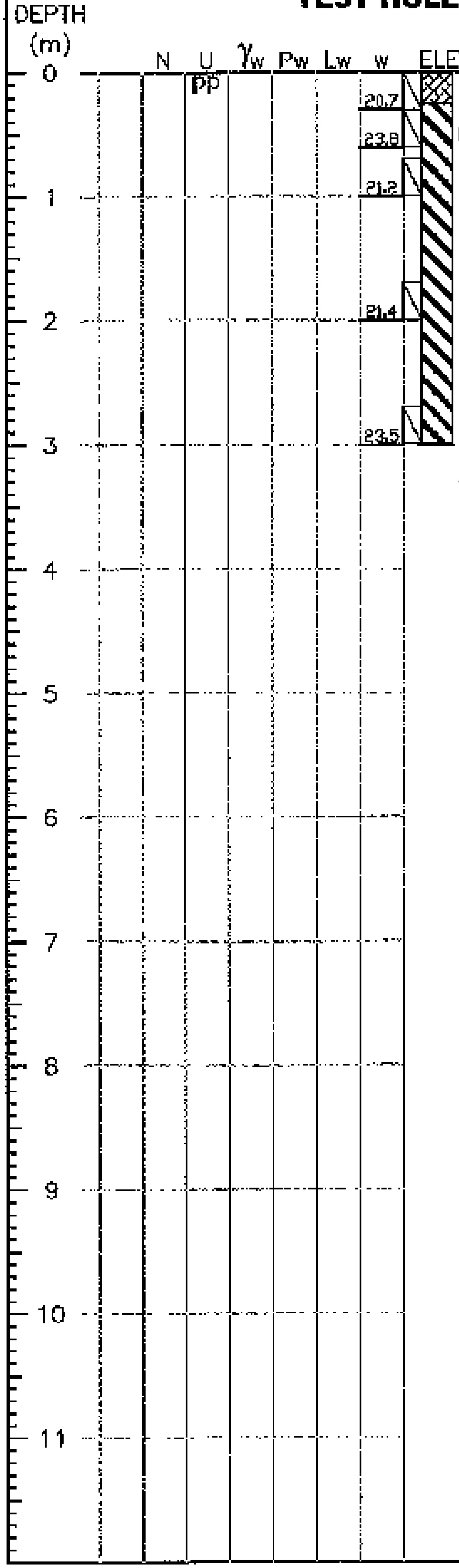
LOCATION:
WITHIN NW-32-56-6-W5M
LAC STE ANNE COUNTY
NEAR SANGUDO, AB

NORTHING: 5972864 **EASTING:** 0640142

DATE DRILLED: NOV 14/14

DRAWING NUMBER: 9769-2

TEST HOLE 14-2



TOPSOIL, black, grass rootlets.

CLAY, some silt, stiff, highly plastic, moist, grey.

NOTE:
1. Test Hole open to 3.0 m and dry I.A.D.

LEGEND:

▩	☰	▦	▒	▥	▧	☒
TOPSOIL	FILL	GRAVEL	SAND	SILT	CLAY	GLACIAL TILL

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

Lw...LIQUID LIMIT

Pw...PLASTIC LIMIT

γ_w...WET UNIT WEIGHT (kN/m³)

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

pp...POCKET PENETROMETER (kg/cm²)

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

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

P200...% PASSING No. 200 SIEVE

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

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

▼...RECORDED WATER LEVEL (PIEZO)

■	☒	⧅
SHELBY TUBE	SPLIT SPOON	CUTTINGS

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

P. MACHIBRODA ENGINEERING LTD.

FIELD DRILL LOG AND SOIL TEST RESULTS

PROJECT:
PROPOSED ADMINISTRATION BUILDING

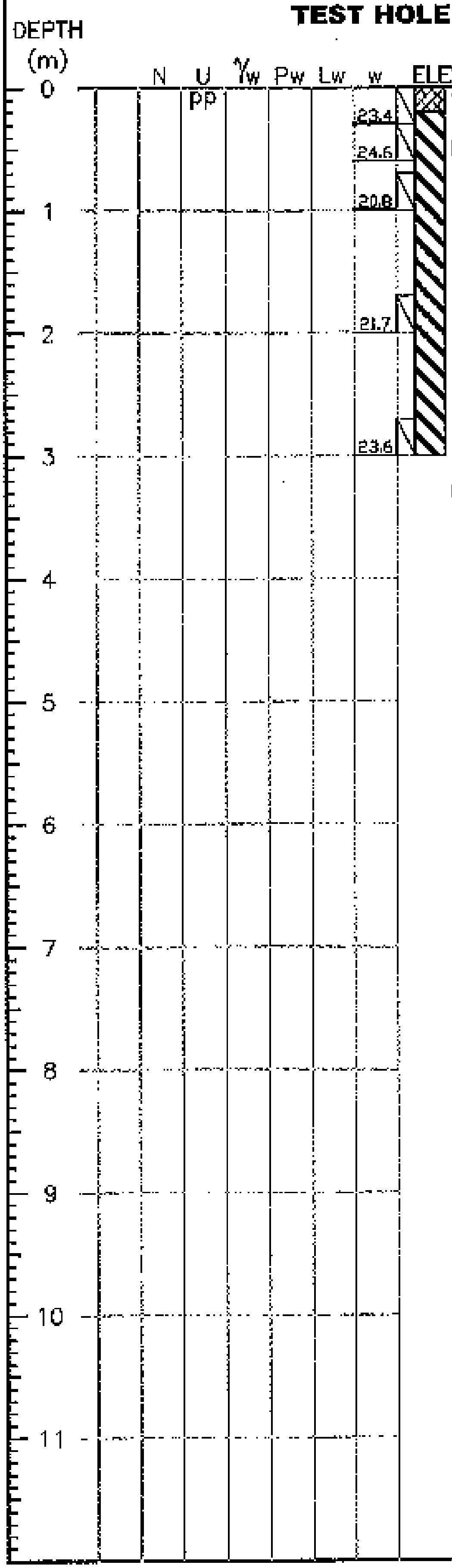
LOCATION:
WITHIN NW-32-56-6-W5M
LAC STE ANNE COUNTY
NEAR SANGUDO, AB

NORTHING: 6972831 **EASTING:** 640103

DATE DRILLED: NOV 14/14

DRAWING NUMBER: 9769-3

TEST HOLE 14-3



TOPSOIL, black, grass rootlets.
CLAY, some silt, stiff, highly plastic, moist, grey, mottled brown.

NOTE:
1. Test Hole open to 3.0 m 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 ADMINISTRATION BUILDING

LOCATION:
WITHIN NW-32-56-6-W5M
LAC STE ANNE COUNTY
NEAR SANGUDO, AB

NORTHING: 5972769 **EASTING:** 6410138

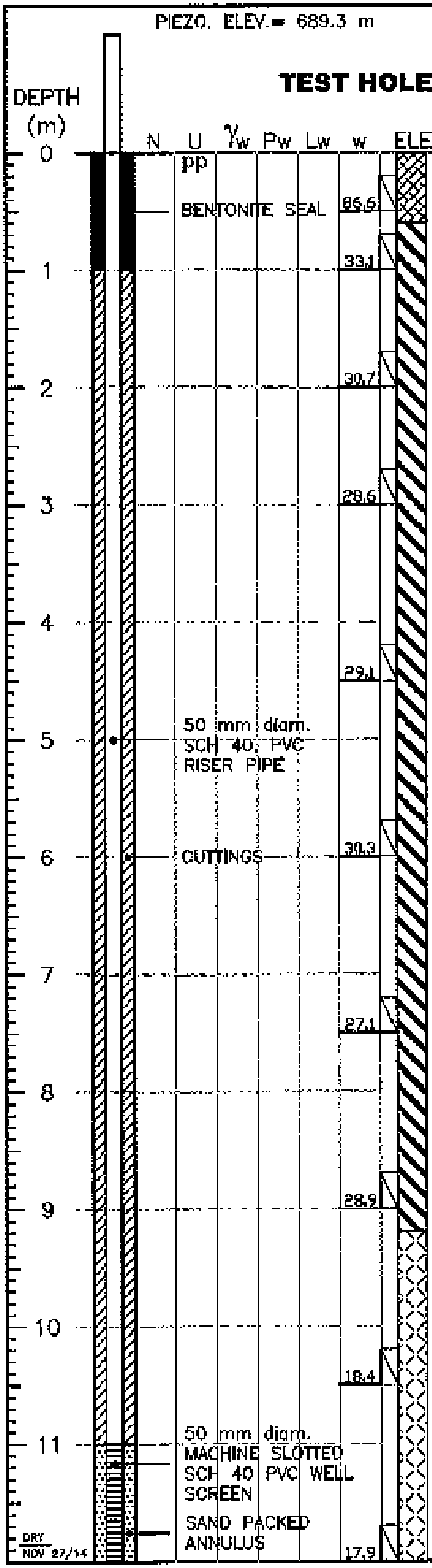
DATE DRILLED: NOV 14/14

DRAWING NUMBER: 9769-4

TEST HOLE 14-4

PIEZO. ELEV.= 689.3 m

ELEV: 688.3 m



TOPSOIL, organics, peat, some clay, soft, moist, black.

CLAY, firm, highly plastic, moist, grey, oxide stained.

–silty, soft, medium plastic between 2.7 to 3.3 m.

–stiff below 5.0 m.

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

NOTE:
1. Test Hole open to 12.0 m 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])

SO₄.....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 ADMINISTRATION BUILDING

LOCATION:
WITHIN NW-32-56-6-W5M
LAC STE ANNE COUNTY
NEAR SANGUDO, AB

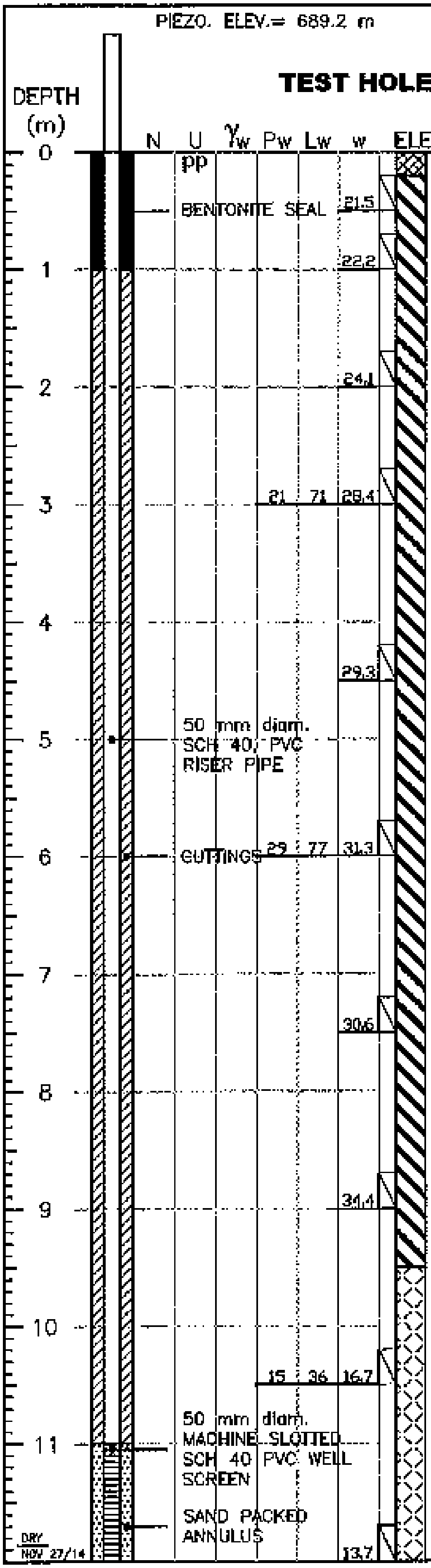
NORTHING: 5972849 **EASTING:** 640165

DATE DRILLED:	**DRAWING NUMBER:**
NOV 14/14	9769-5

PIEZO. ELEV.= 689.2 m

TEST HOLE 14-5

ELEV: 688.0 m



TOPSOIL, black, grass rootlets.

CLAY, some silt, stiff, highly plastic, moist, grey, oxide stained.
—some organics to 600 mm.

—brown between 2.6 to 3.1 m.

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

NOTE:
1. Test Hole open to 12.0 m and dry I.A.D.

LEGEND:

▨ TOPSOIL	☰ FILL	GRAVEL	SAND	SILT	▧ CLAY	GLACIAL TILL

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

Lw...LIQUID LIMIT

Pw...PLASTIC LIMIT

γ_w...WET UNIT WEIGHT (kN/m³)

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

pp...POCKET PENETROMETER (kg/cm²)

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

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

P200...% PASSING No. 200 SIEVE

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

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

▼...RECORDED WATER LEVEL (PIEZO)

■ SHELBY TUBE ☒ SPLIT SPOON ⧄ CUTTINGS

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

P. MACHIBRODA ENGINEERING LTD.

FIELD DRILL LOG AND SOIL TEST RESULTS

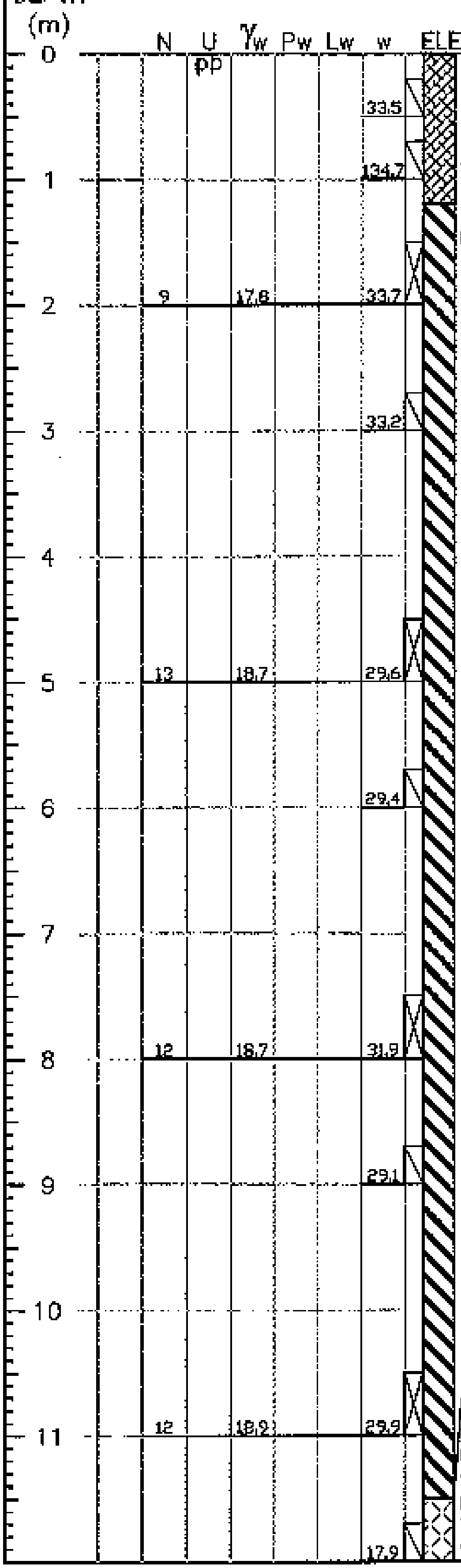
PROJECT:
PROPOSED ADMINISTRATION BUILDING

LOCATION:
WITHIN NW-32-56-6-W5M
LAC STE ANNE COUNTY
NEAR SANGUDO, AB

NORTHING: 5972808 **EASTING:** 640115

DATE DRILLED:	DRAWING NUMBER:
NOV 14/14	9769-6

TEST HOLE 14-6



TOPSOIL, organics, peat, some clay, soft, moist, black.

—some clay below 900 mm.

CLAY, some silt, stiff, highly plastic, moist, grey, gypsum crystals.

GLACIAL TILL, clay, some silt, trace sand, trace gravel, stiff, medium plastic, moist, grey, gyosum crystal, oxide stained.

NOTE:
1. Test Hole open to 12.0 m and dry I.A.D.

LEGEND:

TOPSOIL	FILL	GRAVEL	SAND	SILT	CLAY	GLACIAL TILL
▨	☰	⁂	░	▥	▧	☒

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

Lw...LIQUID LIMIT

Pw...PLASTIC LIMIT

γ_W...WET UNIT WEIGHT (kN/m³)

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

pp...POCKET PENETROMETER (kg/cm²)

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

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

P200...% PASSING No. 200 SIEVE

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

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

▼...RECORDED WATER LEVEL (PIEZO)

■	☒	⧅
SHELBY TUBE	SPLIT SPOON	CUTTINGS

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

P. MACHIBRODA ENGINEERING LTD.

FIELD DRILL LOG AND SOIL TEST RESULTS

PROJECT:
PROPOSED ADMINISTRATION BUILDING

LOCATION:
WITHIN NW-32-56-6-W5M
LAC STE ANNE COUNTY
NEAR SANGUDO, AB

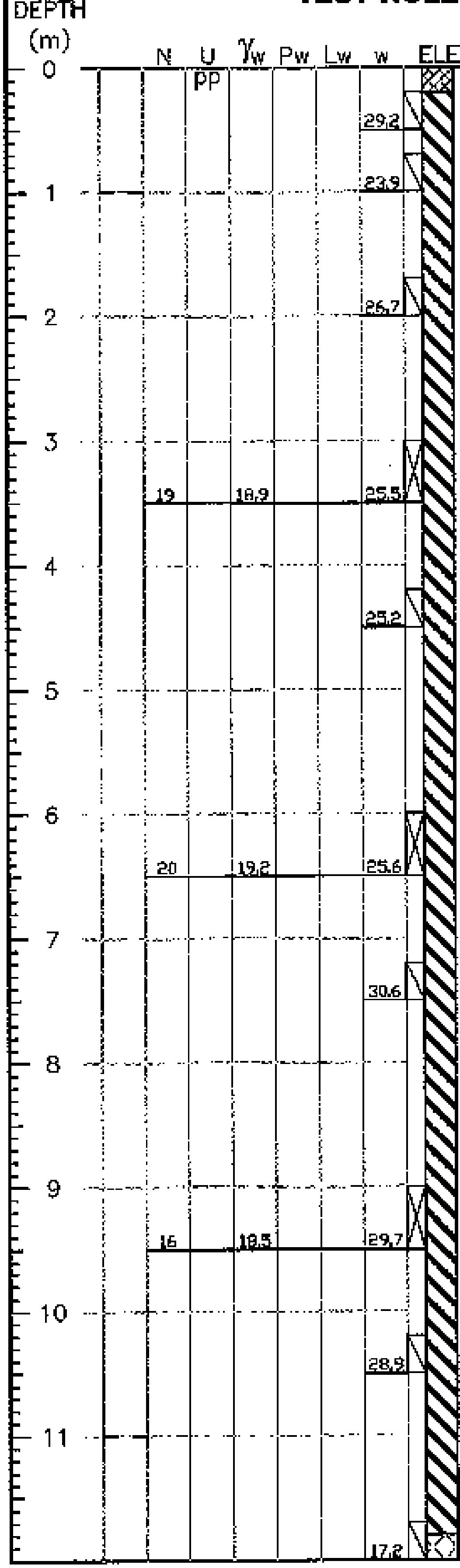
NORTHING: 5972803 **EASTING:** 640161

DATE DRILLED: NOV 14/14

DRAWING NUMBER: 9769-7

TEST HOLE 14-7

ELEV: 687.2



TOPSOIL, black, rootlets.

CLAY, some silt, stiff, highly plastic, moist, grey, gypsum crystals.

–some organics to 600 mm.

–very stiff below 2.0 m.

–brown between 2.3 m and 5.0 m.

–stiff below 8.0 m.

–no silt below 9.5 m.

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

NOTE:
1. Test Hole open to 12.0 m and dry I.A.D.

LEGEND:

TOPSOIL FILL GRAVEL SAND SILT CLAY GLACIAL TILL

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

Lw...LIQUID LIMIT

Pw...PLASTIC LIMIT

γ_w...WET UNIT WEIGHT (kN/m³)

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

pp...POCKET PENETROMETER (kg/cm²)

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

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

P200...% PASSING No. 200 SIEVE

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

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

▼...RECORDED WATER LEVEL (PIEZO)

■ SHELBY TUBE ☒ SPLIT SPOON ⧅ CUTTINGS

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

P. MACHIBRODA ENGINEERING LTD.

FIELD DRILL LOG AND SOIL TEST RESULTS

PROJECT:
PROPOSED ADMINISTRATION BUILDING

LOCATION:
WITHIN NW-32-56-6 W5M
LAC STE ANNE COUNTY
NEAR SANGUDO, AB

NORTHING: 6972852 **EASTING:** 640118

DATE DRILLED: NOV 14/14

DRAWING NUMBER: 8769-8

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.

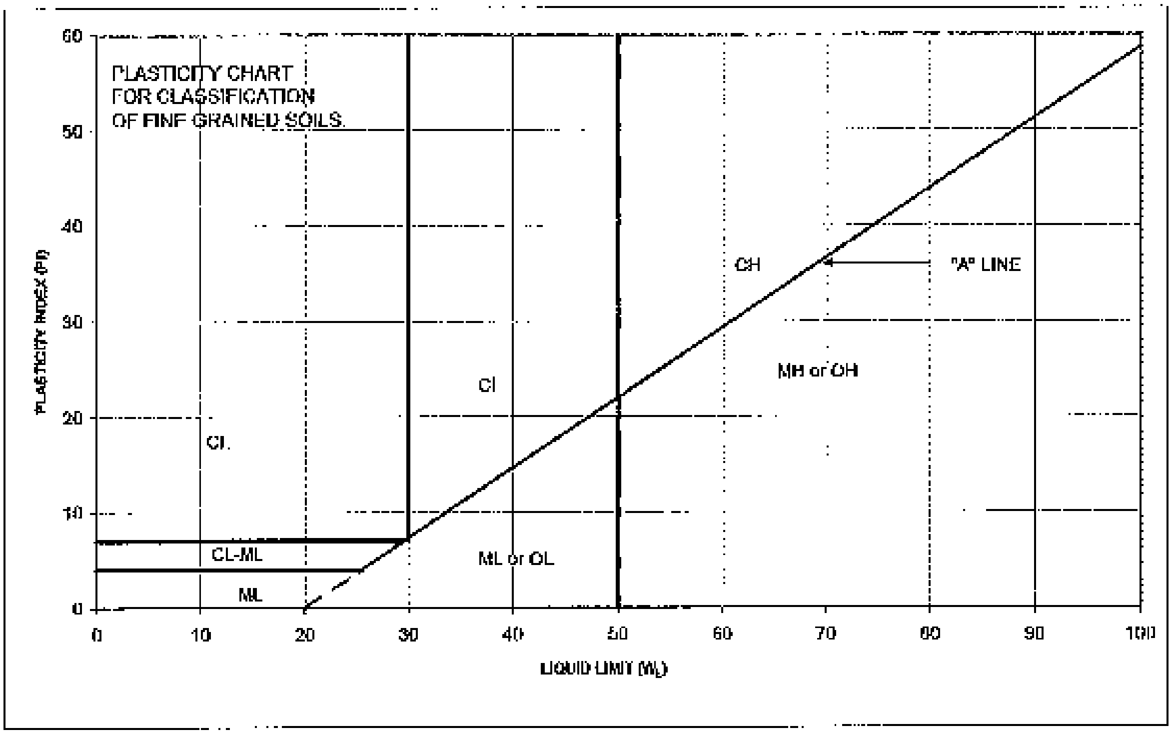
Organic Soils: Readily identified by colour, odour, spongy feel and frequently by fibrous texture.

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	$W_L > 50$



APPENDIX B

LABORATORY TEST RESULTS

ASTM D422: GRAIN SIZE ANALYSIS OF SOIL

Project: PROPOSED ADMINISTRATION BUILDING
WITHIN NW-32-56-6-W5M, LAC STE. ANNE COUNTY, NEAR SANGUDO, AB

Project No.: 9769

Date Tested: NOVEMBER 19, 2014

Test Hole No.: 5

Sample No.: 29

Depth (m): 3.0

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	100
# 20	0.85	100
# 40	0.425	99.6
#60	0.25	99.4
# 100	0.15	98.4
# 200	0.075	97.1

Hydrometer Analysis:

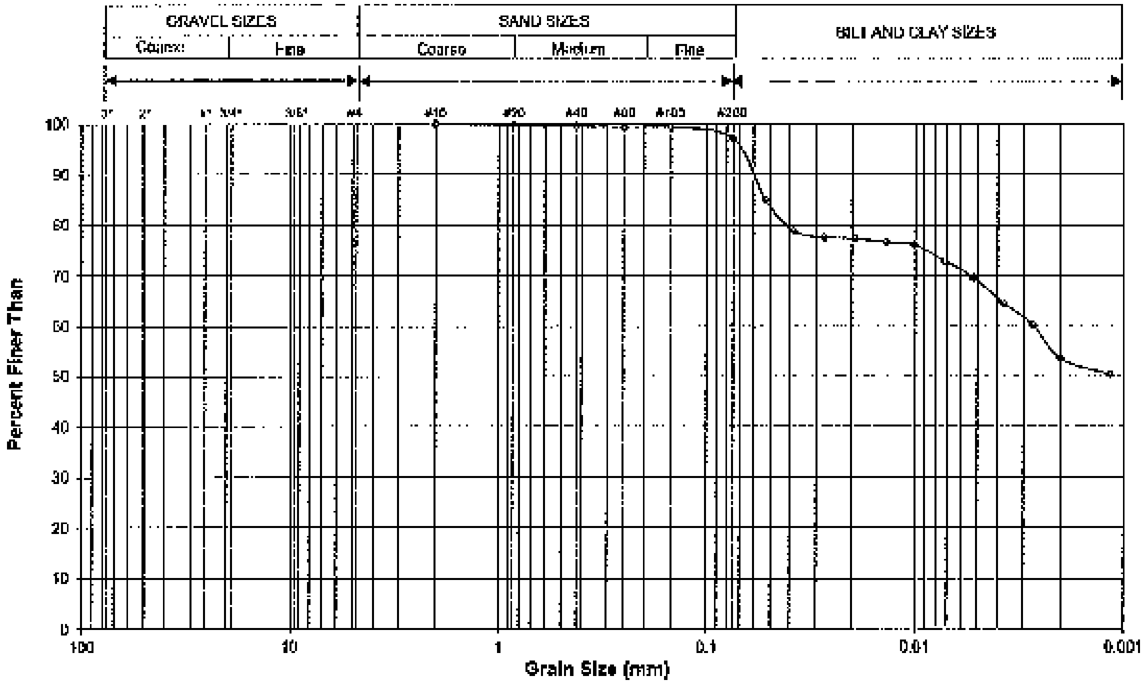
Dispersing Agent:
Sodium Hexametaphosphate

Diameter mm	% Finer
0.0527	85.0
0.0385	78.8
0.0274	77.6
0.0194	77.3
0.0138	76.7
0.0101	76.0
0.0072	72.9
0.0052	69.5
0.0038	64.6
0.0027	60.2
0.0020	53.5
0.0012	50.4

Material Description:

% Gravel Sizes	% Sand Sizes	% Silt Sizes	% Clay Sizes
0	3	44	53

Remarks:



P. MACHIBRODA ENGINEERING LTD.

DRAWING NO.

APPENDIX B-1

ASTM D422: GRAIN SIZE ANALYSIS OF SOIL

Project: PROPOSED ADMINISTRATION BUILDING
WITHIN NW-32-56-6-W5M, LAC STE. ANNE COUNTY, NEAR SANGUDO, AB

Project No.: 9769

Date Tested: NOVEMBER 19, 2014

Test Hole No.: 5

Sample No.: 31

Depth (m): 6.0

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	100
# 20	0.85	100
# 40	0.425	100.0
#60	0.25	100.0
# 100	0.15	100.0
# 200	0.075	100.0

Hydrometer Analysis:

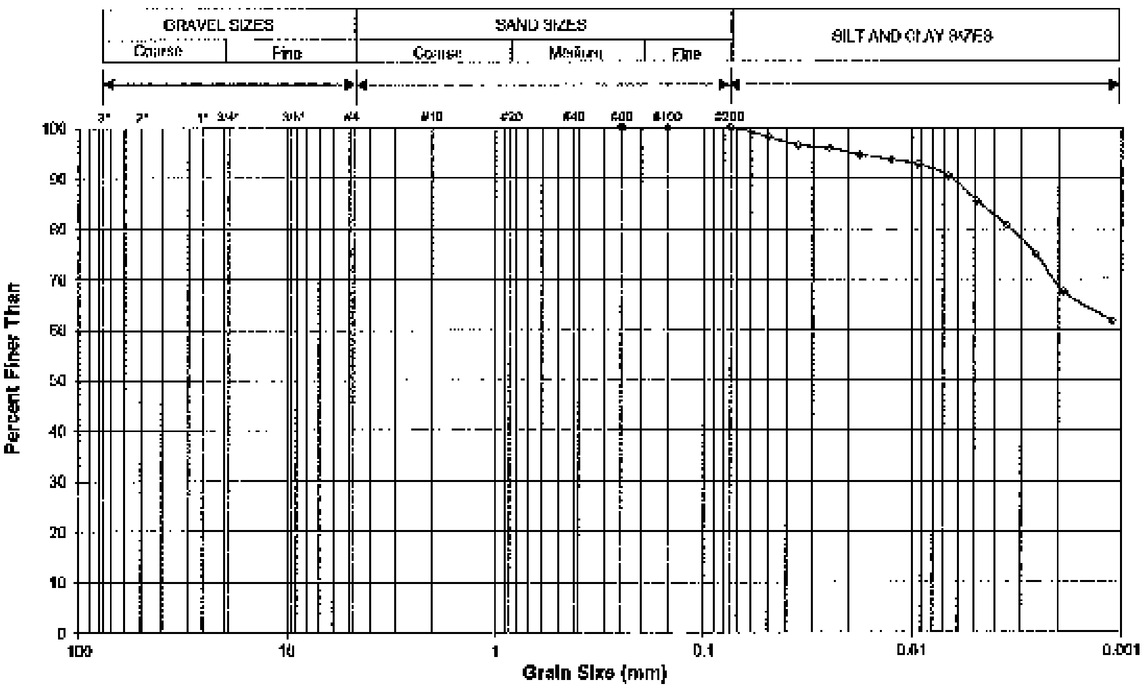
Dispersing Agent:
Sodium Hexametaphosphate

Diameter mm	% Finer
0.0494	98.2
0.0353	96.6
0.0250	96.0
0.0178	94.7
0.0127	93.7
0.0093	92.8
0.0067	90.6
0.0048	85.5
0.0035	80.7
0.0026	75.0
0.0019	67.5
0.0011	61.7

Material Description:

% Gravel Sizes	% Sand Sizes	% Silt Sizes	% Clay Sizes
0	0	32	68

Remarks:



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APPENDIX B-2

ASTM D422: GRAIN SIZE ANALYSIS OF SOIL

Project: PROPOSED ADMINISTRATION BUILDING
WITHIN NW-32-56-6-W5M, LAC STE. ANNE COUNTY, NEAR SANGUDO, AB

Project No.: 9769

Date Tested: NOVEMBER 19, 2014

Test Hole No.: 5

Sample No.: 34

Depth (m): 10.5

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	100
# 20	0.85	97
# 40	0.425	93.8
#60	0.25	86.5
# 100	0.15	77.5
# 200	0.075	68.0

Hydrometer Analysis:

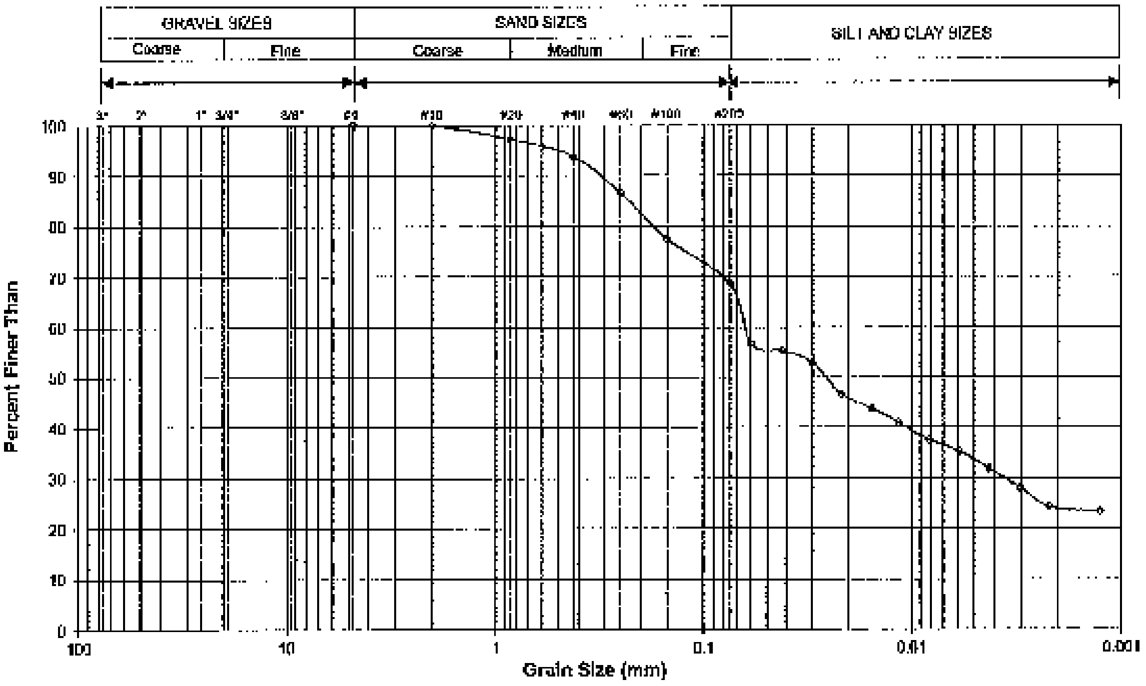
Dispersing Agent:
Sodium Hexametaphosphate

Diameter mm	% Finer
0.0580	56.6
0.0420	55.2
0.0300	52.9
0.0218	46.7
0.0156	43.8
0.0115	41.0
0.0083	37.6
0.0059	35.3
0.0042	31.9
0.0030	28.2
0.0022	24.3
0.0013	23.4

Material Description:

% Gravel Sizes	% Sand Sizes	% Silt Sizes	% Clay Sizes
0	31	44	25

Remarks:



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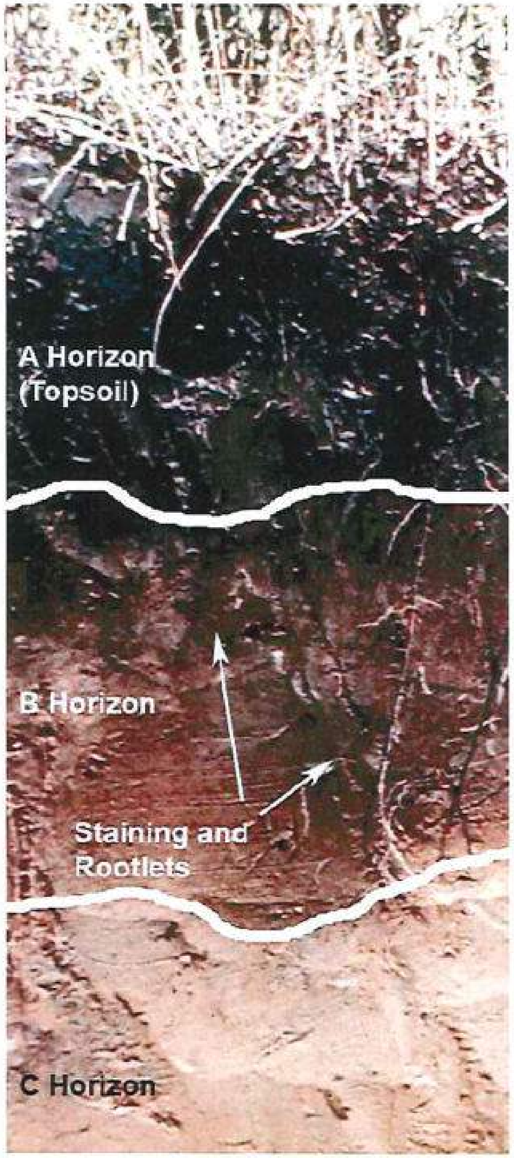
APPENDIX B-3

APPENDIX C

TOP SOIL, 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.