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GEOTECHNICAL INVESTIGATION
PROPOSED SHORED EXCAVATION
RCMP HANGAR - PRINCE ALBERT AIRPORT

190 VETERANS WAY
PRINCE ALBERT, SASKATCHEWAN
PMEL FILE NO. 10121
NOVEMBER 30, 2015

PREPARED FOR:

EGE ENGINEERING LTD.
511 PEPPERLOAF CRESCENT
WINNIPEG, MANITOBA
R3R 1E6

ATTENTION: MR. ANDREW PASSALIS

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1.0 INTRODUCTION

The following report has been prepared on the foundation soil conditions existing at the site of the proposed Shored Excavation to be constructed at the RCMP Hangar located at the Prince Albert Airport - 190 Veterans Way, Prince Albert, Saskatchewan.

Written authorization (via e-mail) to proceed with this investigation was provided on October 19, 2015. The terms of reference for this investigation were presented in P. Machibroda Engineering Ltd. (PMEL) Proposal No. 10121, dated March 10, 2015.

The field investigation was undertaken on November 4, 2015.

2.0 FIELD INVESTIGATION

Three test holes, located as shown on the Site Plan, Drawing No. 10121-1, were dry drilled using our truck-mounted, continuous flight, solid and hollow stem auger drill rig. The test holes were 150 to 200 mm in diameter and extended to depths of 10.5 to 11 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.

Both disturbed and relatively undisturbed soil samples were recovered during test drilling. Relatively undisturbed soil samples were recovered by hydraulically pressing thin-walled Shelby tubes into the bottom of the test hole as test drilling progressed. The Shelby tube samples were sealed to minimize moisture loss. Disturbed samples of auger cuttings, collected during test drilling, were sealed in plastic bags to minimize moisture loss. The soil samples were taken to our laboratory for analysis.

Standard penetration tests (N-index), utilizing a safety hammer with automatic trip were performed during test drilling.

3.0 FIELD DRILL LOGS

The field drill logs recorded during test drilling have been shown plotted on Drawing Nos. 10121-2 to 4, inclusive.

The ground surface elevation at each Test Hole location was referenced to the top of the floor slab of the existing Hangar as shown on the Site Plan, Drawing No. 10121-1. A datum elevation of 100.000 metres was assumed for the top of the slab.

3.1 Soil Profile

In general, the subsurface soil conditions consisted of sod cover followed by a thin layer of fill or silt to depths of about 0.9 to 1.3 metres below grade. Sand was encountered beneath the upper soils and extended to depths of approximately 3.8 to 4.9 metres, at which depth deposits of medium to highly plastic clay and low to medium plastic silt were encountered to the maximum depth drilled with our test holes (i.e., 11 metres below existing ground surface).

The sand was fine to medium grained, poorly graded and loose to compact in density. The underlying clay and silt were predominantly stiff in consistency with an estimated undrained shear strength in the order of 50 to 75 kPa.

3.2 Groundwater Conditions, Sloughing

Groundwater seepage and sloughing conditions were encountered in the sand layer in each test hole below a depth of approximately 2.9 to 3.5 metres below existing ground surface. Hollow stem auger was utilized during test drilling to maintain an open hole through this zone. The depths at which groundwater seepage and sloughing conditions were encountered have been shown plotted on the field drill logs, Drawing Nos. 10121-2 to 4, inclusive.

Groundwater monitoring conducted by EGE Engineering Ltd. at the site revealed the groundwater table to be situated between 3.3 to 3.4 metres below existing ground surface on November 4, 2015. Higher groundwater conditions could be encountered, particularly during or following spring thaw or periods of precipitation.

3.3 Cobblestones and Boulders

Cobblestones and boulders were not encountered within the depths explored with our test holes at this site.

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, unit weights, grain size distribution analysis and triaxial compressive strength testing (UU).

The results of the soil classification, index tests and compressive strength testing 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. 10121-2 to 4, inclusive.

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

5.0 DESIGN RECOMMENDATIONS

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

5.1 Design Considerations

It is understood that the proposed excavation will extend to a depth of about 4 to 5 metres below existing ground surface and that temporary shoring will likely be required to limit the lateral extents of the excavation.

The subsurface soil conditions consisted of approximately 4 to 5 metres of sand overlying variable deposits of clay and silt. Groundwater seepage and sloughing conditions were encountered in the sand below the elevation of the groundwater table (i.e., 3 metres below ground surface).

Sloped excavation sidewalls could be considered at the site (i.e., no shoring), but will encounter construction difficulties related to excessive groundwater seepage and sloughing conditions below the elevation of the groundwater table. Dewatering will be required to provide dry working conditions (i.e., well point pumping system) where sloped excavation will be employed. The dewatering system should be designed to lower the groundwater table a minimum of about 0.6 metres below the base of the excavation. Minimum sideslopes of 3H:1V are recommended through damp sand once the groundwater table has been lowered to below the base of the excavation.

Temporary shoring such as steel sheet piling or steel H-pile and lagging could be considered to limit lateral excavation extents and should perform satisfactorily. Dewatering will be required in order to install the lagging below the elevation of the groundwater table due to flowing conditions anticipated in the saturated sand. Placement of straw or synthetic filter material behind the lagging may be required to prevent loss of soil behind the shoring. Temporary casing will also be required during construction of an H-pile and lagging system to facilitate installation of the H-piles into pre-drilled holes.

Based on the above discussion, steel sheet piling may be a better suited shoring option due to the anticipated construction difficulties associated with the installation of an H-pile and lagging shoring system.

It is also understood that a new above-ground fuel storage tank will be constructed as part of the scope of work for the project. The tank will be constructed within an area that will have contaminated soil excavated and replaced with clean fill. As a result, the tanks will reportedly be supported by piles in order to provide stable foundation support and eliminate potential differential foundation movements as the fill settles. Pile loads are anticipated to be in the order of 150 kN (SLS) and 200 kN (ULS), respectively.

Drilled, cast-in-place concrete piles could be considered for support of the new tank and should perform satisfactorily. Temporary casing will be required to provide an open hole for placement of reinforcing steel and concrete. Alternately, helical screw piles could also be considered and should perform satisfactorily as a foundation support for the new tank.

Recommendations have been prepared in the following sections to assist in the design of the shoring system and excavation at this site. Design recommendations for drilled, cast-in-place concrete piles and helical screw piles have also been provided for the proposed aboveground storage tank.

5.2 Shoring

5.2.1 Cantilever Unbraced Walls

Cantilever (unbraced) shoring systems, such as steel sheet piles or H-piles and lagging could be considered and should perform satisfactorily. As mentioned, flowing conditions are anticipated in the saturated sand and dewatering will be required in order to install wood lagging below the elevation of the groundwater table for H-pile shoring systems. Temporary casing will also be required to facilitate installation of the H-piles into the pre-drilled holes.

Struts or tiebacks could be utilized, if required, to stiffen the shoring system and minimize lateral displacements. The shoring should be designed to withstand lateral earth and hydrostatic pressures as well as surcharge loading.

For cantilever (unbraced) walls, a triangular pressure distribution with the resultant acting at a point one-third (1/3) of the height up measured from the bottom of the excavation should be utilized. The groundwater table should be assumed to be acting at a depth of 3 metres below ground surface for sheet piling shoring systems as it is assumed that any dewatering activities on the inside of the shoring will not lower the groundwater table acting on the shoring significantly.

Dewatering activities for H-pile and lagging systems should lower the groundwater elevation to the base of the excavation and it can be assumed that hydrostatic forces act on the shoring at this elevation. The submerged unit weight of the soil should be used in the calculation of earth pressures below the groundwater table and the hydrostatic pressure distribution should be added to the above pressure distribution. The resultant force may be calculated as:

$$P = \frac{K \quad \gamma \quad H^2}{2}$$

Where

P = Resultant force (kN/m^3)

γ = Unit weight of soil (see Table I)

H = Depth of excavation (m)

K = Coefficient of Soil Pressure

TABLE I. SOIL PARAMETERS FOR LATERAL DESIGN

Soil Type	Effective Angle of Internal Friction	Effective Cohesion (kPa)	Earth Pressure Coefficients		Unit Weight (kN/m^3)	Submerged Unit Weight (kN/m^3)	Undrained Shear Strength (kPa)
			k_a	k_p			
Sand	30°	0	0.3	3.0	18.5	8.5	0
Clay/Silt	23°	5	0.4	2.3	18.5	8.5	65

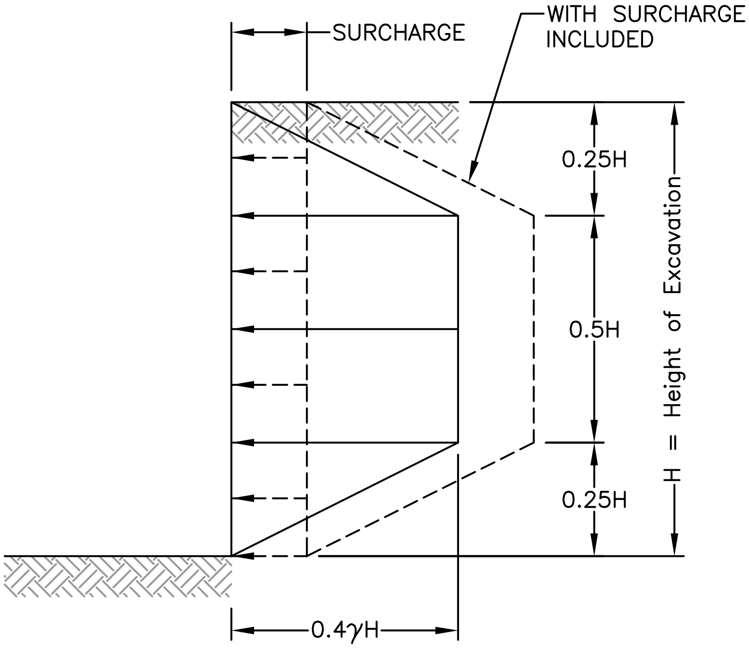
5.2.2 Braced Walls

The distribution of stress against a braced excavation cannot be accurately predicted from earth pressure theory. The apparent earth pressure diagram for a braced excavation is more of a parabolic shape as compared to the triangular distribution for a cantilevered wall. As such, apparent earth pressure diagrams are typically utilized in the design of braced excavations. Based on the soil conditions encountered at this site, a lateral earth pressure distribution diagram for a braced excavation shoring system has been presented in Figure No. 1. The apparent earth pressure diagram should be utilized to estimate the lateral pressure distribution for shoring systems using walers, struts or tiebacks.

5.3 Ground Displacements

Deformation of the shoring system will occur during/after excavation and will cause both vertical and lateral movements within the adjacent ground. The amount of deformation that occurs depends on the flexibility of the shoring system and strength of the supported ground. The soil conditions within the depth of excavation at this site consisted of loose to compact sand and stiff clays. For a flexible shoring system, the maximum lateral and vertical movements of the ground will likely be in the order of 0.5 percent of the wall height, with the greatest displacement occurring adjacent the wall. The zone of influence laterally away from the wall that the soils would undergo displacement is approximately equal in distance to 2 times the height of the wall.

If the above magnitude of displacement is not considered tolerable, the provision of bracing and/or stiffening the shoring system could be considered to lower the anticipated ground movements. If bracing is utilized, the movements are generally small if horizontal bracing is placed as soon as the support level is reached (i.e., typically in the order of 0.1 to 0.3% of the excavation depth).



WHERE: γ = UNIT WEIGHT OF SOIL – (SEE TABLE I)

1) Check system for partial excavation condition.

2) Where the groundwater table is above the base of excavation the submerged unit weight of soil should be used in calculation of earth pressure below the groundwater table and the hydrostatic pressure distribution should be added to the above pressure distribution.

3) If surcharge loads are present at or near the ground surface and are to be carried by walls of the excavation these must be accounted for in the lateral pressure diagram presented above.

NOTE:
1. THIS DRAWING IS FOR CONCEPTUAL PURPOSES ONLY. ACTUAL LOCATIONS MAY VARY AND NOT ALL STRUCTURES ARE SHOWN.

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DRAWING TITLE:
PRESSURE DISTRIBUTION FOR BRACED EXCAVATION

PROJECT:
PROPOSED SHORED EXCAVATION
190 VETERANS WAY, PRINCE ALBERT, SK

SCALE: NOT TO SCALE

DATE: NOVEMBER, 2015

DRAWING NUMBER:
10121-FIGURE NO. 1

5.4 Base Stability

Deep excavations in clay soils have the potential to develop base failures due to removal of overburden pressure and overstressing of the soil at the excavation base. The factor of safety with respect to base heave, FS_b, is as follows:

$$FS_b = N_b s_u / \sigma_z$$

where

N_b = Stability number dependant on the geometry of the excavation (see Figure No. 2, where H = depth & B = width of excavation)

s_u = Undrained shear strength of the soil below the base level of the excavation.

σ_z = total overburden stress acting at the base of the excavation (including surcharge pressures).

Based on the results of the SPT and triaxial compressive strength testing, the estimated undrained shear strength of the clay soils existing below the base of the proposed excavation (i.e., 5 to 6 metres below grade) is in the order of approximately 65 kPa. A factor of safety of 2 or greater is recommended to protect against base heave.

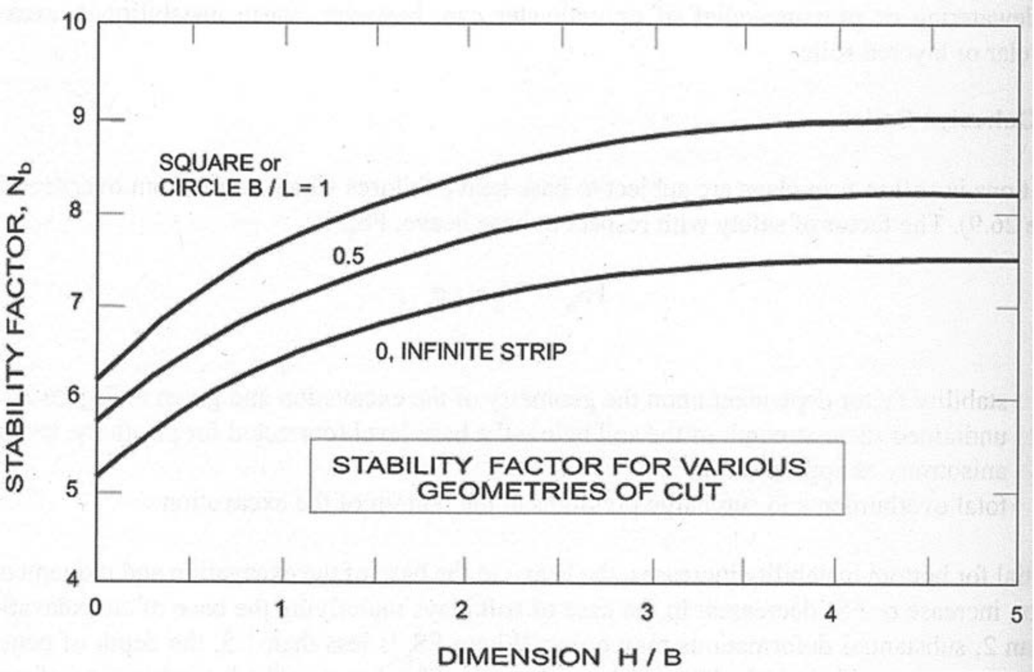

Figure No. 2 Factor of Safety With Respect to Base Failure in Cohesive Soils (CFEM 2006, after Janbu, 1954)

5.5 Sloped Excavations

Temporary excavations at this site should be made in accordance with current Saskatchewan Labour Occupational Health and Safety (OH&S) Guidelines. Based on the results of our geotechnical investigation, the subsurface conditions at this site may be classified as “Type 4” soils. Extensive groundwater seepage and sloughing conditions are anticipated for sand soils below the elevation of the groundwater table. Dewatering (i.e., sand points and/or large diameter wells) will be required in order to lower the groundwater table to below the elevation of the proposed excavation base (minimum of 0.6 metres) if sloped sidewalls will be utilized at this site (i.e., no shoring). Excavations completed in damp sand after dewatering (i.e., Type 3 soils) may be sloped at an angle not steeper than 3H:1V.

The stability of the excavation walls will be affected by wetting and drying of the exposed excavation walls and the length of time that the excavation remains open. To avoid overloading the banks, excavated material should not be placed within a distance equal to 1D from the crest of the slope, where D is defined as the average width of the spoil pile.

5.6 Drilled, Cast-In-Place Concrete Piles

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

The ultimate skin friction bearing pressures of the undisturbed soil have been presented in Table II below. Suitable resistance factors to reduce the provided ultimate skin friction bearing pressures to a value that is suitable for design have been presented in Section 5.9, Limit States Resistance Factors and Serviceability.

TABLE II. SKIN FRICTION BEARING PRESSURES (DRILLED PILES)

Depth (metres)	Ultimate Skin Friction Bearing Pressure (kPa)
0 to 2	0
2 to 5	35
Below 5	45

Notes:

1. Piles installed in recently placed fill material shall be designed to account for negative skin friction. Design recommendations have been presented in Section 5.8.

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

3. Piles should be reinforced.

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

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

6. Groundwater seepage and sloughing conditions were encountered during test drilling and should be expected during pile installation. Temporary casing will be required where groundwater seepage and sloughing conditions are encountered. As casing is extracted, concrete in casing must have adequate head to displace all water in the annular space.

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

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

5.7 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. Suitable resistance factors to reduce the provided ultimate skin friction bearing pressures to a value that is suitable for design have been presented in Section 5.9, Limit States Resistance Factors and Serviceability.

TABLE III. SKIN FRICTION BEARING PRESSURES (SCREW PILES)

Zone (metres)	Ultimate Skin Friction Bearing Pressure Along Pile Shaft (kPa)
0 to 2	0
2 to 5	25

TABLE IV. END BEARING PRESSURES (SCREW PILES)

Depth (metres)	Ultimate End Bearing Pressure (kPa)
0 to 3 (or fill depth, whichever is greater)	0
3 to 5	550
Below 5	650

Notes:

1. Piles installed in recently placed fill material shall be designed to account for negative skin friction. Design recommendations have been presented in Section 5.8.

2. The minimum embedment depth of the uppermost helix for multi-helix piles should be ≥ 3 metres, the depth of fill or H/D = 5 (whichever is greater), where H = depth to top helix, D = helix diameter.

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

4. For determination of skin friction capacity, the effective shaft length (Leff) 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), Leff = H - D - 2.

5. End bearing capacity may be calculated utilizing the effective soil contact area of the helix (i.e., overall cross-sectional area for the lowest helix, helix area minus shaft area for upper helixes).

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

7. The helical plate shall be normal to the central shaft (within 3 degrees) over its entire length. Multiple helixes (if applicable) should be spaced at increments of the helix pitch to ensure that all helixes travel the same path during installation.

8. Continuous monitoring of the installation torque should be undertaken during installation to determine whether the screw pile has been damaged during installation and to monitor the consistency of the subsurface soils.

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

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

5.8 Negative Skin Friction

Piles installed through fill soils may be subjected to down drag forces (negative skin friction) due to long term settlement of newly placed fill. The magnitude of down drag forces depend on soil type, the effective overburden pressure and the total thickness of compressible soil layers below the top of the pile.

Pile downdrag can be computed using the following expression:

$$Q_n = q_n \, C \, D_n \qquad (1)$$

where

Q_n = total down drag load (kN)
q_n = unit negative skin friction (kPa)
C = pile shaft circumference (m)
D_n = length of pile embedded in settling soil (m)

For fill depths greater than 2 metres, the magnitude of unit negative skin friction along the pile may be taken as 10 kPa.

5.9 Limit States Resistance Factors & Serviceability

Limit states are defined as those conditions under which a structure ceases to fulfill the function for which it was designed (i.e., unsatisfactory performance). In limit states design, two conditions are assessed with respect to performance, these are:

- ultimate limit states (ULS), and
- serviceability limit states (SLS)

Ultimate limit states are concerned with the collapse mechanisms of the structure (i.e., safety), whereas serviceability limit states consider mechanisms that restrict or constrain the intended use, function or occupancy of the structure. A further discussion of the limit states design method is described in the Canadian Foundation Engineering Manual (CFEM, 2006) and the National Building Code of Canada (NBCC, 2010).

As per NBCC - 2010, the following resistance factors may be applied to the ultimate bearing pressures presented in Tables II, III and IV to obtain the factored geotechnical resistance corresponding to ultimate limit states (ULS).

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

To satisfy serviceability limit states, 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.

With respect to SLS and deep foundation design, provided the piles are designed using the resistance factors presented above and good construction practices are followed, the amount of settlement of a deep foundation at the design load will be small and within tolerable limits (within the range of 5 to 15 mm).

Drilled straight shaft piles derive their capacity from skin friction and would undergo less movement at the design load (i.e., 5 to 8 mm) as compared to helical screw piles. Helical screw piles are predominantly end bearing piles and would undergo more movement at the design load as compared to predominantly skin friction pile types (i.e., 10 to 15 mm).

The above is applicable to individual piles and small pile groups. Foundation settlement should be evaluated where large pile groups are employed to carry the foundation load (i.e., breadth of foundation or pile cap is a similar dimension as depth of piles).

6.0 LIMITATIONS

The presentation of the summary of the field drill logs and shoring design recommendations has been completed as authorized. Three, 150 to 200 mm diameter test holes were dry drilled using our 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 locations should be anticipated. If conditions should differ from those reported here, then we should be notified immediately in order that we may examine the conditions in the field and reassess our recommendations in the light of any new findings.

The 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 EGE Engineering Ltd. and their agents for specific application to the proposed Shored Excavation to be constructed at the RCMP Hangar located at the Prince Albert Airport - 190 Veterans Way, Prince Albert, Saskatchewan. It has been prepared in accordance with generally accepted geotechnical engineering practices and no other warranty, express or implied, is made.

Any use which a Third Party makes of this report, or any reliance on decisions to be made based on it, is the responsibility of such Third Parties. 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. Should you require additional information, please contact us.

P. MACHIBRODA ENGINEERING LTD.

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KP/TW:tbs:ldw

Association of Professional Engineers &
Geoscientists of Saskatchewan
CERTIFICATE OF AUTHORIZATION
P. MACHIBRODA ENGINEERING LTD.
Number 172
Permission to Consult held by:

Discipline	Sk. Reg. No.	Signature
Geotechnical	10461	K. Pardoski 15-11-30

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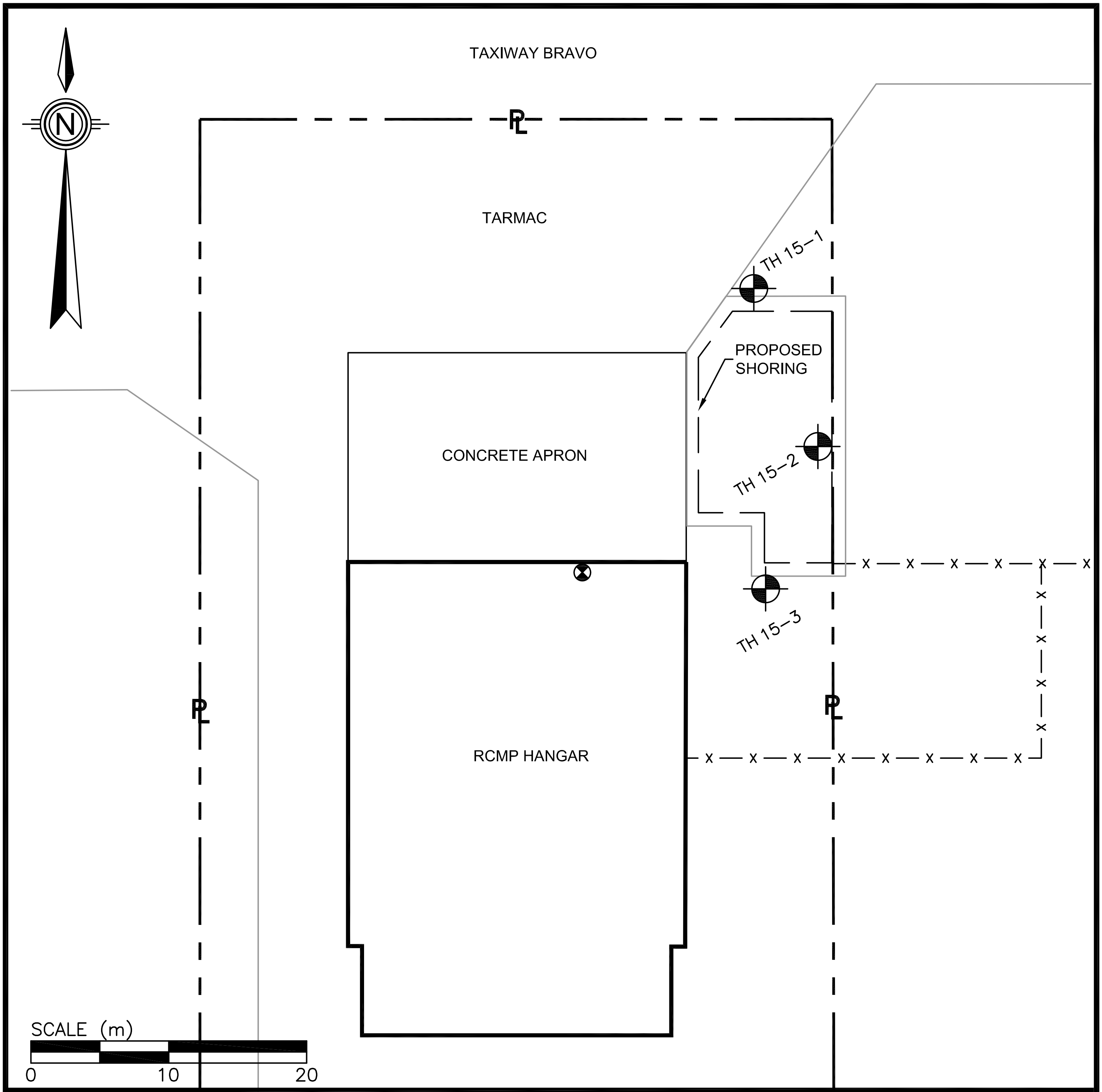
DRAWINGS



NOTE:
1. THIS DRAWING IS FOR CONCEPTUAL PURPOSES ONLY. ACTUAL LOCATIONS MAY VARY AND NOT ALL STRUCTURES ARE SHOWN.
2. BENCHMARK: FLOOR SLAB OF RCMP HANGAR. ASSUMED DATUM ELEVATION = 100.000 m.

LEGEND	⊕ –PMEL TEST HOLE	⊗ –BENCHMARK	PL –PROPERTY LINE

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DRAWING TITLE:
SITE PLAN - TEST HOLE LOCATIONS

PROJECT:
PROPOSED SHORED EXCAVATION
190 VETERANS WAY, PRINCE ALBERT, SK

APPROVED BY: KP

DRAWN BY: TP

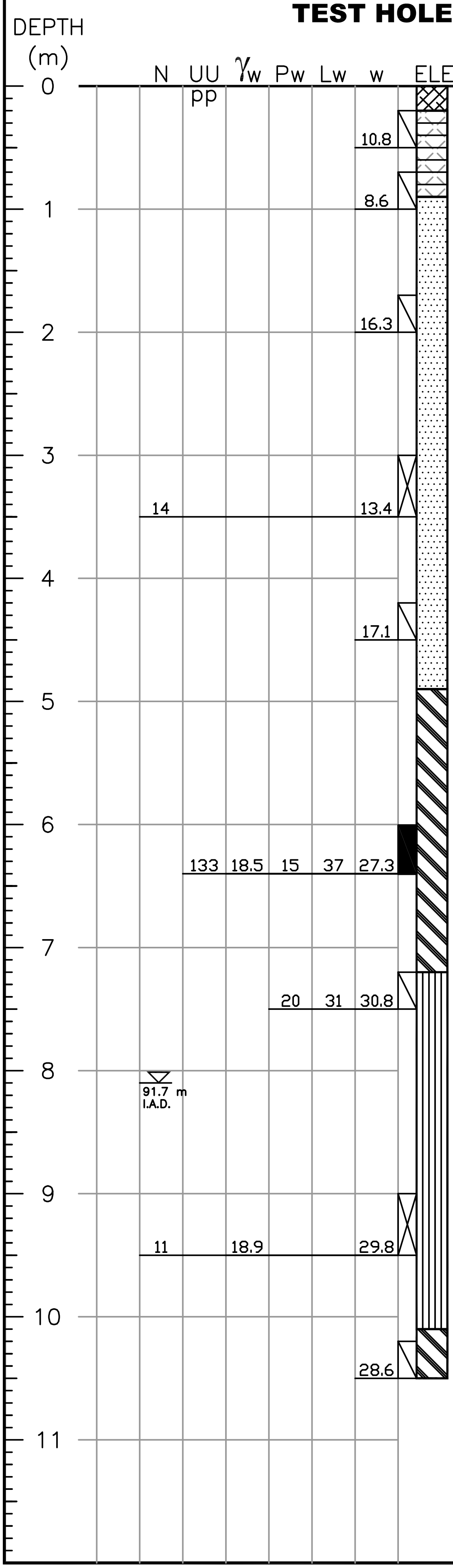
DRAWING NUMBER: 10121-1

DATE: NOVEMBER, 2015

SCALE: AS SHOWN

TEST HOLE 15-1

ELEV: 99.8 m



TOPSOIL, moist, black, rootlets.

FILL, glacial till, clay, some silt, some sand, trace gravel, low to medium plastic, moist, brown.

SAND, silty, loose to compact, poorly graded, fine grained, damp, brown.

–moist below 2.0 m.

–medium grained, grey below 3.4 m.

–wet, seepage, sloughing below 3.5 m.

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

SILT, some clay, some sand, firm, low to medium plastic, moist, grey.

–stiff below 8.9 m.

CLAY, silty, stiff, highly plastic, moist, grey.

NOTE:
1. Hollow stem installed to 6.0 m.
2. Backfilled with benonite chips from 10.5 to 4.0 m.
3. Cuttings from 4.0 m to ground surface.

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)

UU.....TRIAXIAL 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 SHORED EXCAVATION

LOCATION:
190 VETERANS WAY
PRINCE ALBERT, SK

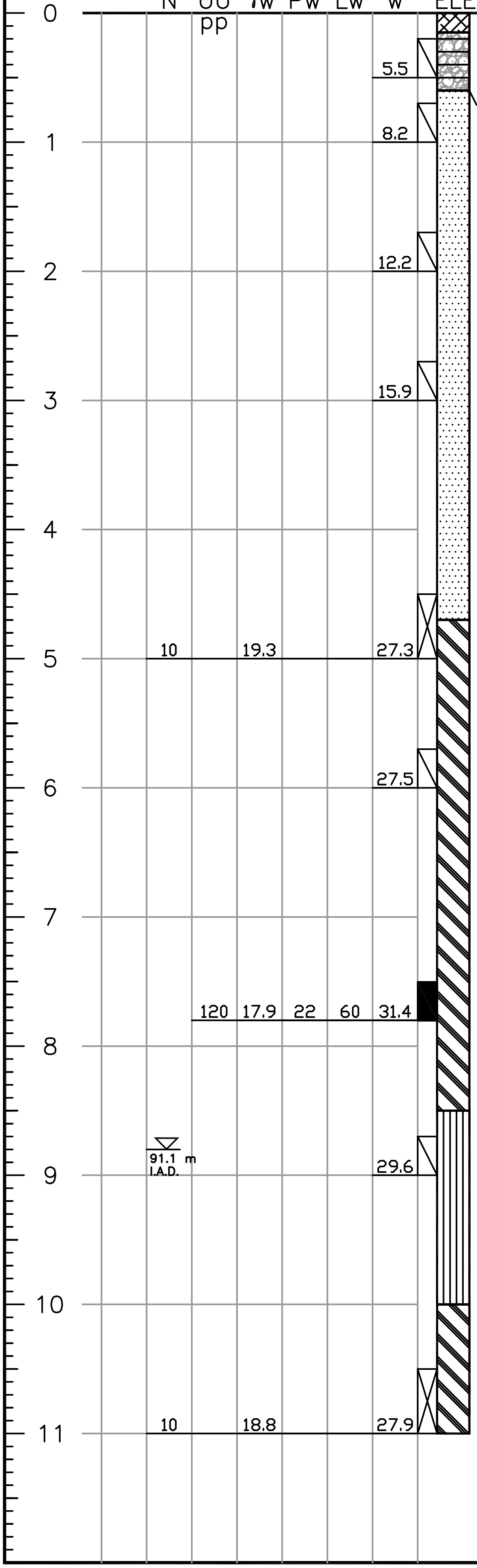
NORTHING:

EASTING:

DATE DRILLED: NOV 4/15

DRAWING NUMBER: 10121-2

TEST HOLE 15-2



N	UU pp	γw	Pw	Lw	w
					5.5
					8.2
					12.2
					15.9
10		19.3			27.3
					27.5
	120	17.9	22	60	31.4
					29.6
10		18.8			27.9

TOPSOIL, moist, black, rootlets.

FILL, gravel and sand, compact, well graded, coarse grained, damp, brown.

SAND, trace silt, loose to compact, poorly graded, fine grained, damp, brown.

–moist below 2.0 m.

–medium grained, grey, wet, seepage, sloughing below 2.9 m.

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

SILT, sandy, trace clay, stiff, low to medium plastic, moist, grey.

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

NOTE:
1. Hollow stem installed to 6.0 m.
2. Backfilled with benonite chips from 10.5 to 4.0 m.
3. Cuttings from 4.0 m to ground surface.

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)

UU.....TRIAXIAL 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 SHORED EXCAVATION

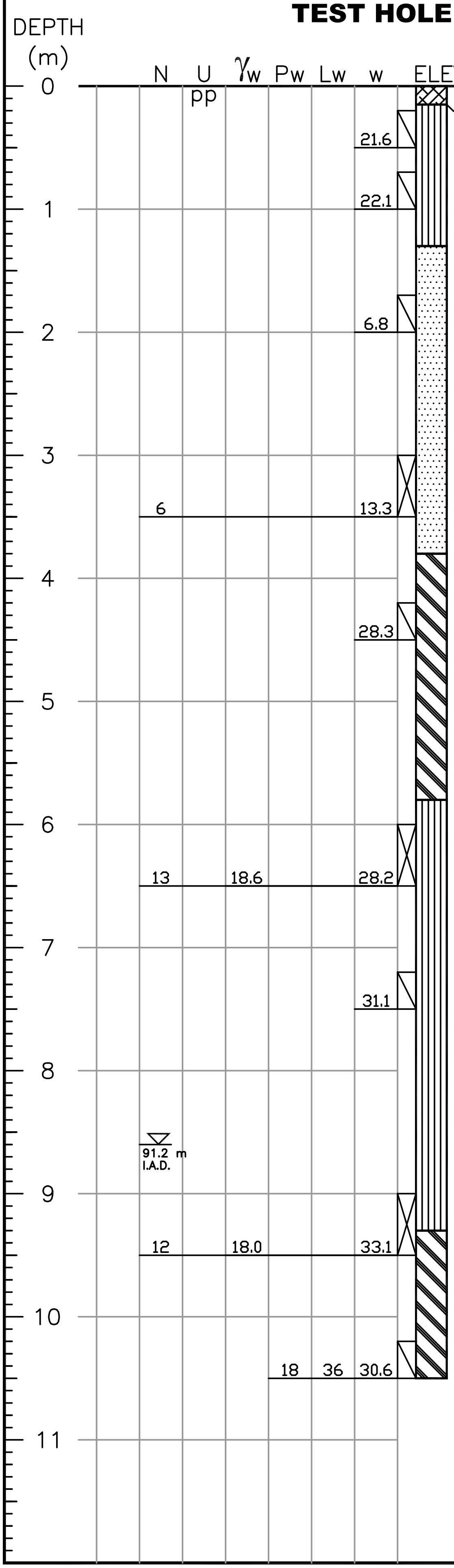
LOCATION:
190 VETERANS WAY
PRINCE ALBERT, SK

NORTHING: **EASTING:**

DATE DRILLED: NOV 4/15

DRAWING NUMBER: 10121-3

TEST HOLE 15-3



TOPSOIL, moist, black, rootlets.

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

SAND, loose to compact, poorly graded, fine grained, damp, brown.

–medium grained below 2.2 m.

–wet, grey, seepage, sloughing below 3.3 m.

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

SILT, trace clay, trace sand, firm, low plastic, moist, grey.

–clayey below 6.4 m.

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

NOTE:
1. Hollow stem installed to 6.0 m.
2. Backfilled with benonite chips from 10.5 to 4.0 m.
3. Cuttings from 4.0 m to ground surface.

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 SHORED EXCAVATION

LOCATION:
190 VETERANS WAY
PRINCE ALBERT, SK

NORTHING:

EASTING:

DATE DRILLED: NOV 4/15

DRAWING NUMBER: 10121-4

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.

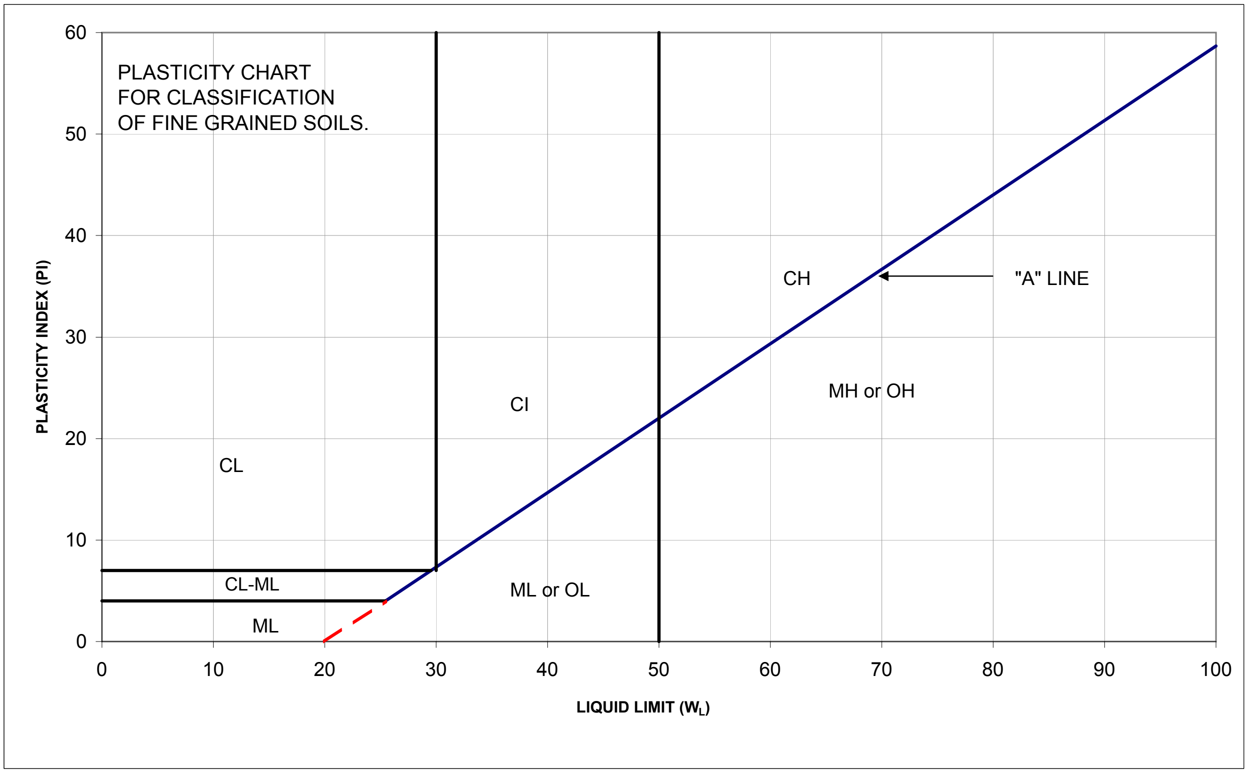
Organic Soils: Readily identified by colour, odour, spongy feel and frequently by fibrous texture.

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

GRAIN SIZE DISTRIBUTION TEST REPORT

Project: Proposed Shored Excavation
190 Veterans Way, Prince Albert, SK

Project No.: 10121

Date Tested: November 10, 2015

Test Hole No.: 15-1

Sample No.: 3

Depth (m): 2.0

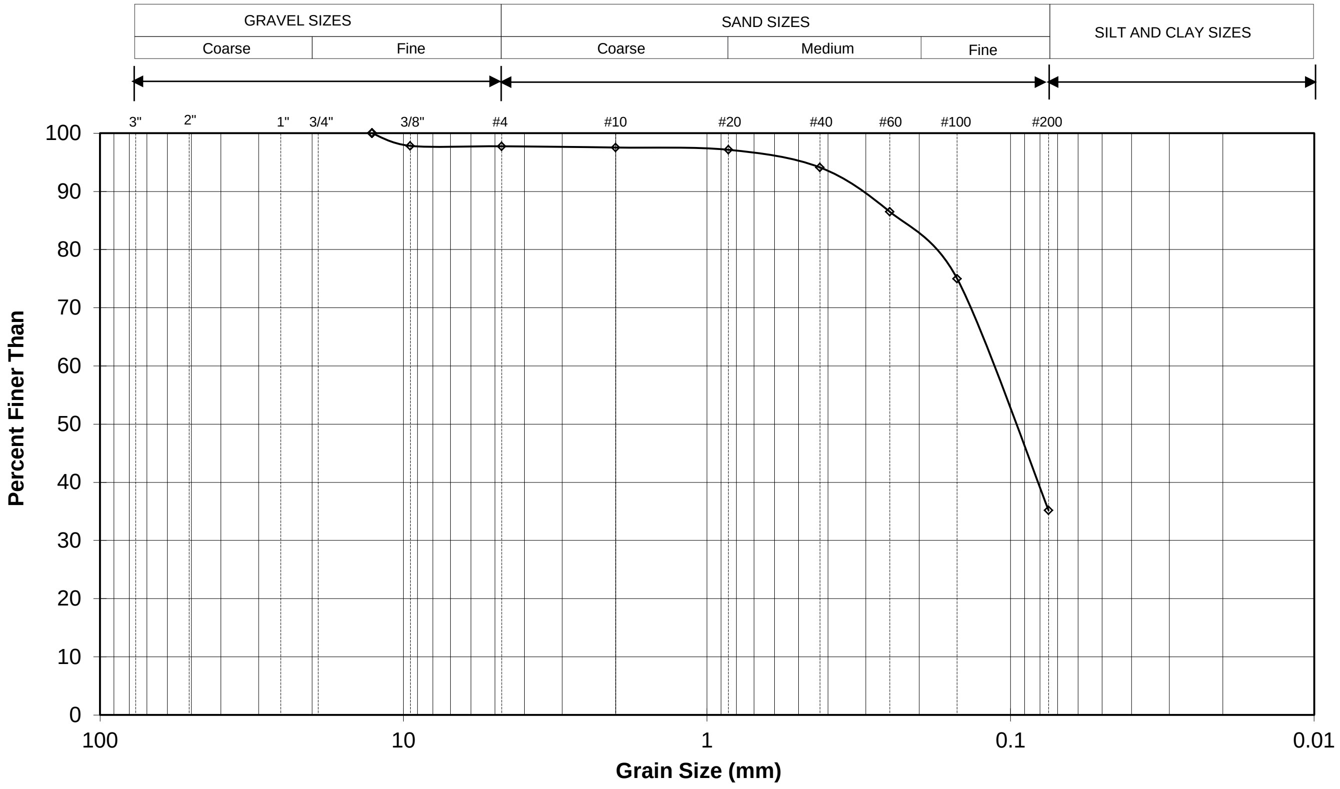
Sieve Analysis:

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	98
# 4	4.75	98
# 10	2	98
# 20	0.85	97
# 40	0.425	94
#60	0.25	86
# 100	0.15	75
# 200	0.075	35

Material Description:

% Gravel Sizes	% Sand Sizes	% Silt and Clay Sizes
2	63	35

Remarks:



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DRAWING NO.

Appendix B-1

ASTM D422: GRAIN SIZE ANALYSIS OF SOIL

Project:	Proposed Shored Excavation
	190 Veterans Way, Prince Albert, SK
Project No.:	10121
Date Tested:	November 16, 2015
Test Hole No.:	15-1
Sample No.:	6
Depth (m):	6-6.4

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	99.9
# 100	0.15	99.8
# 200	0.075	85.6

Hydrometer Analysis:

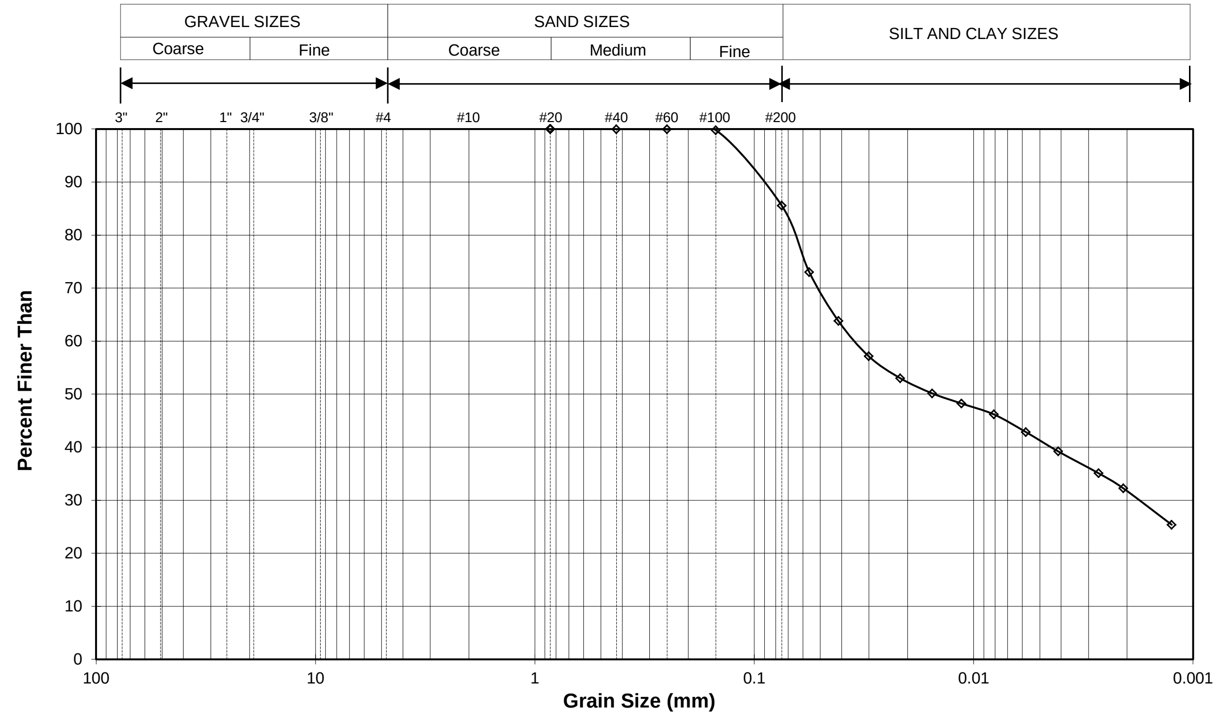
Dispersing Agent:
Sodium Hexametaphosphate

Diameter mm	% Finer
0.0562	73.0
0.0414	63.8
0.0301	57.1
0.0216	53.0
0.0155	50.2
0.0114	48.2
0.0081	46.2
0.0058	42.9
0.0041	39.2
0.0027	35.1
0.0021	32.2
0.0013	25.4

Material Description:

% Gravel Sizes	% Sand Sizes	% Silt Sizes	% Clay Sizes
0	14	53	33

Remarks:



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Appendix B-2

GRAIN SIZE DISTRIBUTION TEST REPORT

Project: Proposed Shored Excavation
190 Veterans Way, Prince Albert, SK

Project No.: 10121

Date Tested: November 10, 2015

Test Hole No.: 15-2

Sample No.: 13

Depth (m): 3.0

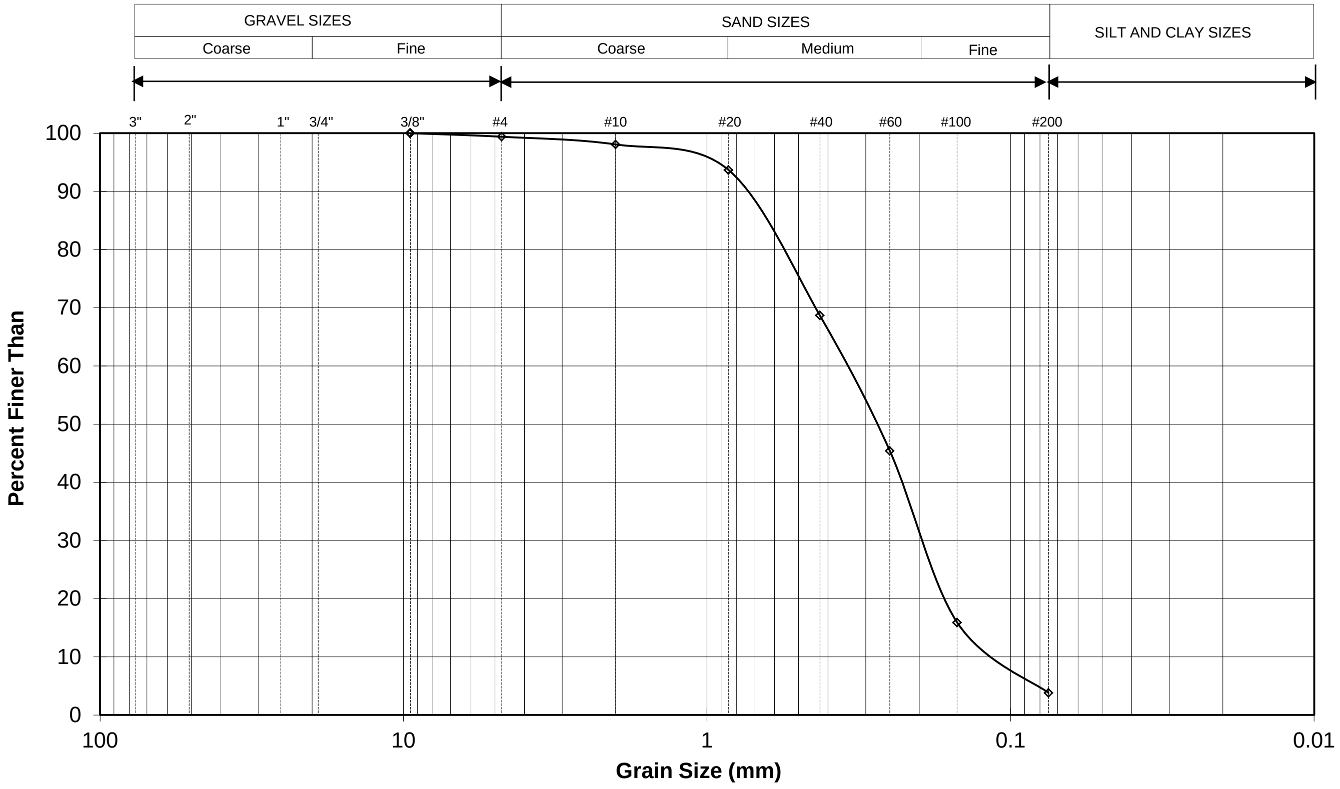
Sieve Analysis:

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	99
# 10	2	98
# 20	0.85	94
# 40	0.425	69
#60	0.25	45
# 100	0.15	16
# 200	0.075	4

Material Description:

% Gravel Sizes	% Sand Sizes	% Silt and Clay Sizes
1	96	3

Remarks:



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Appendix B-3

ASTM D422: GRAIN SIZE ANALYSIS OF SOIL

Project:	Proposed Shored Excavation 190 Veterans Way, Prince Albert, SK
Project No.:	10121
Date Tested:	November 16, 2015
Test Hole No.:	15-2
Sample No.:	16
Depth (m):	7.5-7.8

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	99.9
# 200	0.075	99.7

Hydrometer Analysis:

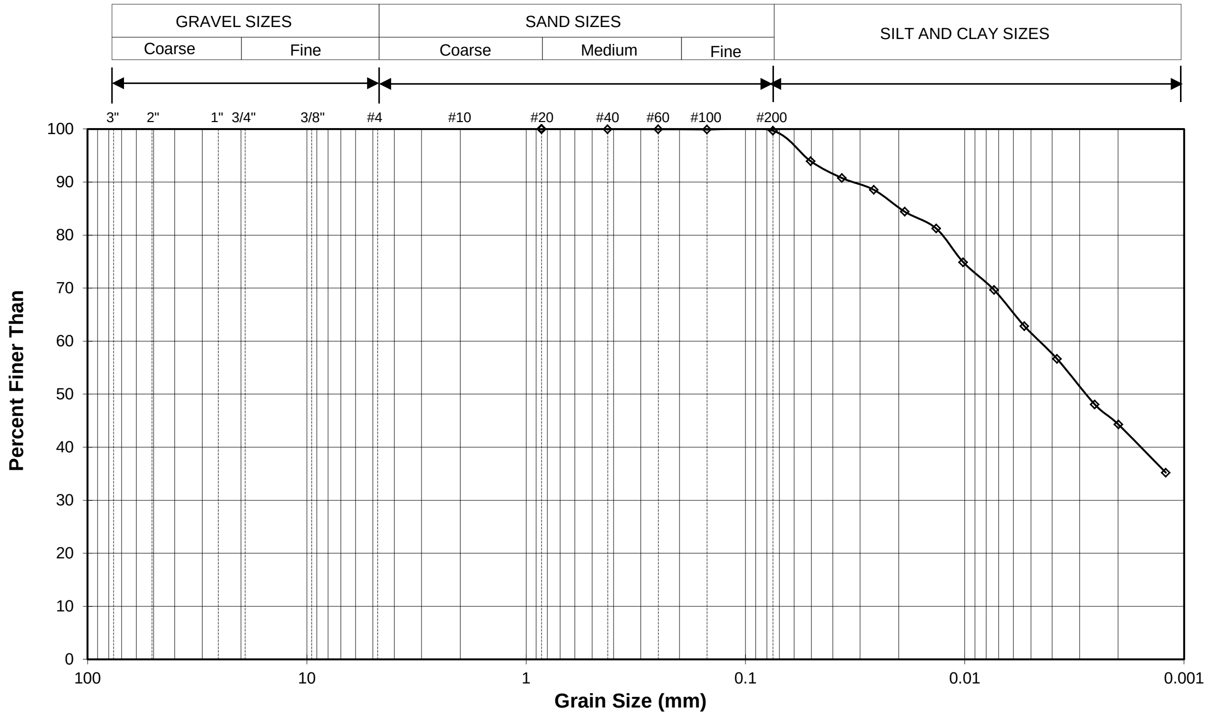
Dispersing Agent:
Sodium Hexametaphosphate

Diameter mm	% Finer
0.0504	93.9
0.0363	90.8
0.0260	88.5
0.0188	84.4
0.0135	81.2
0.0102	74.9
0.0074	69.7
0.0053	62.8
0.0038	56.6
0.0026	48.1
0.0020	44.3
0.0012	35.2

Material Description:

% Gravel Sizes	% Sand Sizes	% Silt Sizes	% Clay Sizes
0	0	55	45

Remarks:



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Appendix B-4