Environmental Geotechnical and Materials Engineering
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GEOTECHNICAL INVESTIGATION

SANDY POINT DEVELOPMENT

BLOCK 9, PLAN 982 4270
LACOMBE COUNTY, ALBERTA

PREPARED FOR

DELTA LAND COMPANY INC.
SYLVAN LAKE, ALBERTA

PREPARED BY

PARKLAND GEOTECHNICAL CONSULTING LTD.

RED DEER, ALBERTA

PROJECT No.: RD3773

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1.0 INTRODUCTION

Delta Land Company Inc. is proposing to construct a marina on the southwest shore of Gull Lake, within SE 1-41-1-W5M in Lacombe County, Alberta. The site location is shown on the Key Plan, Figure 1. Parkland Geotechnical Consulting Ltd. (ParklandGEO) was requested to perform a geotechnical investigation of the site for the proposed marina. The scope of work was outlined in ParklandGEO's proposal dated March 4, 2011 (File# PRO2085REV). Authorization to proceed with this investigation was given by Mr. Lance Dzaman of Delta Land Company Inc. This report summarizes results of field and laboratory testing programs and presents geotechnical recommendations for the proposed marina. Geotechnical recommendations are provided with respect to siting of the marina structure and site grading.

2.0 SITE AND PROJECT DESCRIPTION

The site was located on the southwest shore of Gull Lake, north of Highway 12 on RR1-1in Lacombe County, Alberta. The legal address of the site was Block 9, Plan 982 4270. The Site Plan is shown on Figure 2. The site was snow covered and forested at the time of the investigation. The site slopes to the southeast toward the lake. The elevation difference between survey points ranged from 879 and 885 m. The site was accessed from Range Road 1-1 to the west of the site. The surrounding land use is agricultural lands to the north, west, south and Gull Lake to the east. The Town of Bentley was located about 4 km to the southwest and Aspen Beach Provincial Park about 3.5 km southeast of the site. The present site development and vegetation at the site is shown on the Aerial Photograph provided in Figure 3.

The project consists of a new paved road andmarina with canal, boat launch and parking area. The road will be the primary roadway into the new RV park and will connect to the marina through a gate. The road is expected to be subject to light to moderate vehicle traffic. The marina will be about 195 m in length and 68 m in width and will contain 156 boat docks. It is understood that H-piles with concrete inserts are proposed for the perimeter walls at the marina. A parking area will be constructed to the northwest and a 2 lane boat launch will be located on the northeast. A 30 m wide canal will be dredged on the northeast side of the marina to a depths of 1.5 m. A 30 m minimum environmental reserve setback will be established between the shoreline and the marina.

3.0 FIELD AND LABORATORY PROGRAMS

On March 14, 2011, four deep boreholes were drilled within the proposed marina footprint and roadway to depths between 5.0 and 10.5 m below existing grade. Three shallow probes were drilled in the proposed canal and shoreline to a depth of 3.0 m below grade. The locations of these boreholes are shown on Figure 2. The soil encountered was visually examined during drilling and logged according to the Modified Unified Soil Classification System. Soil samples were taken at 1.0 m intervals in order to determine the soil/moisture profile. Standard Penetration Tests were performed in the deep boreholes at selected depth intervals. All soil samples were returned to ParklandGEO's Red Deer laboratory for selected soil testing to determine soil properties.

At the completion of drilling, standpipes were installed in all deep boreholes. Groundwater levels were recorded on completion and on March 23, 2011. The local groundwater surface elevations were surveyed by ParklandGEO and referenced to a geodetic datum.

4.0 SOIL CONDITIONS

The general soil profile was typical for this area, consisting of in descending order: sand, silty clay, glacial till and bedrock. The detailed soil conditions encountered at the borehole locations are described on the borehole logs in Appendix A. The soil test results and definitions of the terminology and symbols used on the borehole logs are provided on the explanation sheets also in Appendix A. The following is a brief description of the soil types encountered.

4.1 SAND

Surficial sand was observed in Boreholes 3, 5 and 6 with a thickness of 0.3 m. A sand deposit was observed in Borehole 2 and extended beyond the depth drilled (ie. > 3.0 m). The sand was fine to medium grained, poorly graded and silty. The sand generally gets siltier with depth. The non plastic sand was in a loose to compact state, brown to grey with a moisture content ranging from 25 to 43 percent. These sand deposits were considered to be a relatively stable subgrade material. However, like all fine grained sands, this sand will be sensitive to disturbance if encountered in a wet to saturated condition.

4.2 ORGANICS

Organics were encountered in Boreholes 4 and 6 with a thickness of 1.3 and 0.2 m respectively. The soil was moderately organic, black, moist to saturated. The organic content of the soil in Borehole 4 was found to be 8.0 percent which is considered to be low.

4.3 SILTY CLAY

A layer of silty clay was encountered below the surficial sand and buried organics in Borehole 6 at a depth of 0.5 m and extended to a depth of 2.5 m. The clay deposit was medium plastic with a stiff consistency. The moisture contents ranged from 37 to 41 percent. Based on local experience, the Optimum Moisture Content (OMC) of these deposits are estimated to be about 16 to 20 percent. Therefore, the soil moisture content of these deposits are considered to be well above OMC. Due to fine grain size distribution, these silty deposits were considered to be highly frost susceptible and sensitive to disturbance when wet.

4.4 TILL

Glacial clay (till) was encountered in all boreholes except Borehole 2. The till was encountered at or near surface in Boreholes 1, 3, 5, 7 and below the organics in Borehole 4 and extended beyond the depths drilled (ie. > 3-5m). In Boreholes 6 and 7, the till extended to depths of 4.6 and 4.4 m, respectively. The till was a homogeneous mixture of silt, sand and clay with trace gravel, and occasional rust stains, coal inclusions and water bearing sand lenses. The till was medium plastic, stiff to very stiff, with a moisture content ranging from 17 to 24 percent. Based on local experience, the OMC of the clay till is about 15 percent. Therefore, the soil moisture contents of the till are considered to be at or slightly above OMC.

4.5 BEDROCK

Weathered siltstone and sandstone was encountered below the till in Boreholes 6 and 7 beginning at depths of 4.6 and 4.4 m, respectively. This corresponds to an elevation of about 875 m. The local bedrock is considered to be a weak rock with the relative density of a very dense soil. The bedrock becomes more competent with depth.

4.6 WATER SOLUBLE SULPHATES

Soil samples were taken at a depth of 2.0 m for water soluble sulphate concentration testing in Boreholes 6 and 7. The concentrations of sulphates are expressed as a percentage of the dry mass of soil. The concentrations of water soluble sulphate were 0.04, which indicates a "negligible potential for sulphate attack on buried concrete in direct contact with soil."

5.0 GROUNDWATER

Groundwater seepage was observed in Boreholes 3, 6 and 7 during drilling. The following table summarizes the observed groundwater conditions on March 23, 2011, about 8 days after drilling.

TABLE 1
GROUNDWATER MEASUREMENTS

BH #	Ground Elevation (m)	Groundwater Level at Completion (m)	Groundwater Level on Mar 23, 2011 (mbg)	Groundwater Elevation on Mar 23, 2011 (m)
1	-	Dry	1.83	-
3	881.43	4.2	1.26	880.17
6	879.82	3.0	1.06	878.76
7	880.07	3.0	1.07	879.00

The groundwater levels are expected to be close to the lake water elevations due to the proximity of the boreholes to Gull Lake. Given the time of the site investigation, the observed groundwater and lakewater level is considered to be near the seasonal average. Groundwater and lake water elevations are expected to fluctuate higher on a seasonal basis and will be highest after periods of heavy precipitation or snow-melt. The volumes of groundwater encountered will be dependent on seasonal conditions and the size and permeability of clay soil layers.

6.0 DISCUSSION AND RECOMMENDATIONS

6.1 GEOTECHNICAL EVALUATION

The subsurface conditions at this site are considered to be suitable for the construction of the roadway and proposed marina development. It is understood that the marina will be dug out and site grading cut/fills will be undertaken to level and raise areas to smooth out grades at the site and provide access to the marina. The marina canal will be dredge to a depth of 1.5 m.

The soil conditions are considered to be suitable for driven pile systems. The typical driven pile system is driven steel piles. It is understood that H-piles with concrete inserts are proposed for the marina. Foundations recommendations are given for driven steel piles. Information on other foundation options can be provided, upon request. The main geotechnical issues regarding development are:

- That the silty surficial soil is relatively sensitive to disturbance which can result in potential problems during construction depending on actual weather and ground conditions. An observational approach based on the actual conditions at the time of construction is considered the best way to optimize costs by identifying problem areas before construction activity leads to subgrade failure

- That relatively sensitive lacustrine soils may be encountered during site development trenching, depending on where the final grade is set. The subgrade is expected to be sensitive and marginally stable, and like most clays, these materials will be encounter some problems if backfill or deep grading fills are placed during periods of extended wet weather. The clay till deposit was considered to be a suitable material for engineered fill, but moisture conditioning will be required. Wet soils should be mixed or replaced with drier fill or selectively used for general site fill.

- For the roadway and parking area, the subbase of the pavement structure may be placed on a prepared clay subgrade. The level of subgrade support from the upper soils will be low in the silty clay soils. The use of a geo-textile as a separation barrier between the pavement gravel and the fine grained subgrade is strongly recommended to minimize the movement of fines into the gravel base course at all locations. Additional geogrid should be considered for critical traffic areas such as site accesses and internal traffic pathways. Site preparation measures will be significantly impacted by wet weather.

- The siltier surficial soils will be highly frost susceptible if they are given access to free water or groundwater within the zone of seasonal frost (estimated to an average depth of 2.5 m). In general, the depth to the local water table for the site is very shallow (grade level near lake) and within the potential depth of frost.

6.2 SITE PREPARATION

It is anticipated that the marina will be dug out and site grading cut/fills will be undertaken to level and raise areas to smooth out grades at the site around the marina and channel. The channel is to be excavated to 1.5 m in total depth. The finished marina and channel bottom is to be completed using native lake bottom material. All material is to be placed and compacted to ensure soil stability. The channel is to be maintained per Alberta Environment's lake bottom dredging procedures. The exposed subgrade should be proofrolled under the supervision of experienced geotechnical personnel to identify potential soft areas. Soft areas should be sub-cut and replaced with a suitable fill material to a depth sufficient to support construction traffic. If excessively soft subgrade conditions are encountered across this site, preparation procedures should be reviewed. Methods to avoid subgrade failure of soft subgrades may include: limiting construction traffic, modification of site preparation procedures (scarification, recompaction, etc) and sub-cut and replacement with a suitable engineered fill material.

Fill required to bring the site up to grade should be well graded select sand, gravel or low to medium plastic clay. The clay deposits are considered suitable for use as engineered fill, but they will require moisture conditioning in order to achieve specified densities. If coarse gravel is proposed, it is recommended to use a gravel with a maximum aggregate size of 100 mm. A suggested gradation specification is provided in Table 2:

TABLE 2
100 mm COARSE GRAVEL

Sieve Size (mm)	Percent Passing By Weight
100	100
50	85 - 100
25	30 - 70
5	25 - 60
0.080	2 - 10

Fill material should be placed to a uniform density of 98 percent of Standard Proctor Maximum Dry Density (SPMDD). Deeper new fill, including trench backfills more than 1.0 m thick should be placed uniformly to at least 100 percent of SPMDD. . The lift thicknesses should be governed by the ability of the selected compaction equipment to uniformly achieve the recommended density. However, it is generally recommended to use lifts with a maximum compacted thickness of 200 mm for granular fill and 150 mm for clay fill. Uniformity is of most importance

Fills will be subject to some settlement even at high levels of compaction. The estimated settlement under self weight for clay fill placed to 98 percent SPMDD would be 0.5 to 1.0 percent of the fill thickness. For clay fill placed to 95 percent SPMDD the fill settlement would be up to about 1.5 to 2.0 percent of the fill thickness. To minimize the potential for harmful differential settlements, uniformity of fill compaction is most important. For highly compacted clay till and granular fill, the majority of settlement (ie. over 75 percent) is expected to occur during the construction period.

If subgrade conditions are soft, a thicker initial lift may be required to form a working base for subsequent construction. This condition is best addressed in the field at the time of construction. If subgrade conditions warrant the use of subgrade improvement gravel, it is possible, for lower lifts, to use less expensive select coarse gravel with a maximum aggregate size of 150 mm.

6.3 EXCAVATIONS AND BACKFILL

Excavations will be required. The latest edition of the Construction Safety Regulations of the Occupational Health and Safety Act of Alberta should be followed. Excavation side slopes are not expected to be able to stand near vertical for extended periods of time. For excavations deeper than 1.5 m, side slopes should be cut back to 1H:1V. If space does not permit the slopes to be cut back, some form of temporary shoring must be installed to protect workers in the trench.

For excavations through fill or into the groundwater table, flatter side-slopes may be required. If space does not permit the slopes to be cut back, some form of temporary shoring must be installed to protect workers in the trench. All temporary surcharge loads should be kept back from the excavated faces a distance of at least one-half the depth of the excavation. All vehicles delivering materials to the site should be kept back from excavated faces at least 1.0 m.

6.4 LIMIT STATES DESIGN

In accordance with the 2005 National Building Code of Canada (NBCC) and the associated 2006 Alberta Building Code (ABC), the use of Limit States Design (LSD) is required for the design of buildings and their structural components including foundations. The limit states of LSD design are classified into two groups; the Ultimate Limit States (ULS) and the Serviceability Limit States (SLS).

The ULS case is primarily concerned with safety and the levels of load and resistance at the point of collapse or structural failure. The geotechnical value for this case is the ultimate resistance. For foundation design this ultimate resistance value is reduced using a Geotechnical Resistance Factor (GRF) which is based on the reliability index of the geotechnical data used to determine the ultimate resistance for the foundation loading case. As per the NBCC the following GRF values should be used for foundation design for deep foundations:

TABLE 3
LSD GEOTECHNICAL RESISTANCE FACTORS*

GEOTECHNICAL CASE - DEEP FOUNDATIONS (PILES)	Resistance Factors
Vertical resistance by semi-empirical analysis and in-situ test data	0.4
Vertical resistance from analysis of dynamic monitoring results	0.5
Vertical resistance from analysis of static load test results	0.6
Lateral load resistance	0.5

* NBCC - Users Guide - Structural Commentaries (Part 4 of Division B) - Commentary K -Foundations.

The Serviceability Limit State (SLS) occurs when the foundation loads cause movements or vibrations that are greater than the structure can tolerate before the intended use of the structure is restricted or hindered. The SLS case is addressed by determining the maximum available resistance to keep the foundation deformation under service loads (ie. settlement, lateral deflection, etc.) within tolerable limits as provided by the structural engineer. Therefore, the foundation loads, configurations and serviceability tolerances have to be known to properly determine geotechnical SLS resistance values. The tolerable limit of total settlement for foundations subject to compression loads is typically up to 25 mm. For friction piles less than 15 mm of settlement is required to develop full soil pile friction. As a result, the SLS case often does not govern the piled foundation design unless very strict settlement tolerances are required (ie. less than 10 mm of settlement).

Design parameters are provided for ULS design. Specific foundations can be assessed for SLS conditions upon request. SLS analysis will require a full understanding of foundation configuration, loads and settlement/lateral tolerances.

6.5 PILE FOUNDATIONS

6.5.1 Driven Steel Piles - Ultimate Limit States

Corrosion of the pile in a partially saturated medium must be considered in selecting wall thickness. Driven steel piles may be designed using the ultimate resistance values for shaft friction and end bearing provided in the following table.

TABLE 4
DRIVEN STEEL PILES - ULTIMATE RESISTANCE

Soil Type	Depth (m)	Ultimate Resistance (kPa)	
		Skin Friction	End Bearing
Silt/Sand	0 - 2.2	0	-
Till	2.2 - 4.6	65	
Bedrock	4.6 +	125	2250

The ultimate resistance values in this table are based on semi-empirical data, therefore the “factored” resistance should be calculated by multiplying the unfactored values above by a geotechnical resistance factor of 0.4, in accordance with the building code (see Table 3 in Section 6.4). Additional capacity may be available if dynamic monitoring or static load testing is proposed.

The ULS resistance of driven steel piles is determined by multiplying the factored ULS skin friction resistance by the exterior surface area of the pipe pile or the surface area of the web and outside face of the flanges for H-piles. The upper 2.2 m of pile shaft, or the length of pile shaft in new fill, whichever is greater, should be assumed to carry no load. The pile capacity should not exceed the structural capacity for the steel section of the pile. Piles driven through new fills should be assumed to have a down-drag (negative skin friction) equal to 10 kPa for the section of pile shaft within the fill. The minimum depth of pile embedment for resistance to frost action is 7.0 m for unheated areas. Additional recommendations for driven steel piles at the sites are as follows:

1. Steel piles should be driven using maximum hammer energies of 450 to 600 J per square centimetre of pile cross section. For smaller pipe piles a minimum pipe wall thickness of 10 mm is recommended for this site. For pipe piles greater than 500 mm in diameter the minimum pipe wall thickness should be increased to 12.5 mm.

2. Steel piles should not be driven beyond practical refusal. For preliminary purposes, the practical refusal criteria may be taken as 8 blows per each 25 mm interval for the last 300 mm of pile penetration. The actual refusal criteria should be verified once the hammer energies and pile details are known. For steel piles driven to practical refusal prior to achieving design depth, but beyond the required minimum required embedment depth, the allowable load capacity may be determined by multiplying the cross-sectional area of steel at the tip by 0.35 f_y where f_y is yield strength of steel. The maximum permissible value of f_y should be supplied by the manufacturer (nominally 240 MPa).

3. The minimum allowable pile spacing should be taken as three pile diameters. Where groups of piles are to be installed, the piles should be installed starting at the centre with outer piles installed last. The elevations of the tops of piles already installed should be monitored as adjacent piles are driven in order to determine if heaving of the piles has occurred. Piles that have heaved must be re-driven. If groups of piles are installed at a pile spacing less than the minimum, a group reduction factor must be applied to the bearing capacity of each pile.

4. If steel pipe piles are used, it is suggested to fill the piles with concrete after installation. Concrete filling of the open pipe will add strength to the section, reduce the corrosion potential inside the pipe and help facilitate pile cap connections. Corrosion of the pipe in a partially saturated medium must be considered in selecting pipe wall thickness.

5. The steel piles should be inspected prior to installation to confirm that the appropriate material specifications are satisfied; and to check that there are no protrusions on the shaft or at the tip which could result in voids along the shaft as the pile is driven.

6. Monitoring of the pile installation by experienced geotechnical personnel is recommended to confirm that the piles are installed in accordance with design assumptions and that the driving criteria are satisfied. A complete driving record of blows per 300 mm of penetration for each pile should be obtained and reviewed by the pile designer.

7. A Pile Driving Analyzer (PDA) test program should be considered to verify the ultimate pile resistance for this site. For resistance values verified by this dynamic monitoring method the GFR used to calculate the factored resistance may be increased to 0.5, resulting in a 25 percent increase in pile capacity for the ultimate limit states. A static load test program could also be considered to further increase the factored resistance, but given the expected size of this project, a load test is not likely to be cost effective.

6.6 LATERAL LOAD RESISTANCE OF PILES

Piles resist laterally applied loads by deflecting until the necessary resistance is mobilized in the surrounding soils. The load carrying capacity of the soil is determined when: the capacity of the soil is exceeded; excessive bending moments are generated in the pile shaft resulting in structural failure; or the deflections of the pile head are too large for the structure. The design of laterally loaded piles is dependent on the strength of the surrounding soil, the stiffness of the piles, the number of piles in a group, the fixity of the pile cap and the point of load application with respect to the pile/pile cap. The lateral load is generally resisted within the upper 4 to 5 m of the soil profile (ie. the typical point of inflexion for the piles). For preliminary purposes, it is assumed that the lateral capacity of piles will be limited by a deflection criteria of 6 mm or one percent of the pile diameter, whichever is larger. Consideration must be given to ice action on marina walls during the winter months.

The best procedure for determining the lateral load capacity of piles at this site is to perform a lateral load test on a test pile. Alternatively, the theoretical capacity for pile resisting lateral loads may be calculated using one of several available computer models or accepted graphical solutions.

1. For lateral pile resistance, most commercially available pile design packages use the method of p-y curves developed by Reese in 1984 for the Federal Highways Association COM624P computer program[2]. For this method, the strength-deformation characteristics for the various soil layers are modelled by load-displacement curves which vary non-linearly with depth. Standard p-y curves are usually built into the software for a range of typical soils, but some programs allow input of soil specific curves developed from field tests. The design process using these software programs is an iterative procedure to determine deflections and bending moments at given depth increments along the pile shaft for the proposed lateral load and loading condition.

2. As described in the Canadian Foundation Manual, the most common graphical method for determining the resistance of piles against lateral loads and moments is the Method of Broms[3,4]. This method calculates the ultimate capacity for two types of failure: short piles where the lateral capacity of the soil is fully mobilized; and long piles where the bending resistance of the pile is fully mobilized. This method also determines the deflection based on theory of subgrade reaction. Since the majority of the lateral resistance is developed in the near-surface soils, the soil characteristics used in this analysis should be consistent with that of the upper soil deposits. In this case, the upper soil around the piles are expected to be native clay till deposits.

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2 FHWA-IP-84-11

3 Broms, B.B., (1964), Lateral Resistance of Piles in Cohesive Soils, Journal of the Soil Mechanics Division, American Society of Civil Engineering, Vol. 90, SM 2, March, pp. 27-63.

4 Broms, B.B., (1964), Lateral Resistance of Piles in Cohesionless Soils, Journal of the Soil Mechanics Division, American Society of Civil Engineering, Vol. 90, SM 3, May, pp. 123-156.

Most theoretical methods for lateral pile resistance, including the methods discussed above, treat the soil layers around the pile as a series of springs which simulate the elastic reaction of the soil to pile deformation when subjected to horizontal load. The elastic behaviour of the soil can be estimated using an equivalent spring constant known as the Modulus of Subgrade Reaction (k_S). The recommended Modulus of Subgrade Reaction values as a function of the pile diameter (D) are given below:

TABLE 5
MODULUS OF SUBGRADE REACTION FOR LATERAL LOADS ON PILES

Loading Condition	Modulus of Subgrade Reaction, (k_s in MN/m^3)		
	Clay < 2.2 m	Clay Till 2.2 - 4.6 m	Bedrock > 4.6 m
Sustained lateral loads	4.0/D	8.4/D	16/D
Cyclic lateral loads	3.0/D	6.3/D	12/D
Transient lateral loads	5.0/D	10.5/D	20/D

D = Pile Diameter in metres

TABLE 6
SOIL PARAMETERS FOR LATERALLY LOADED PILE ANALYSIS

Soil Parameter	Clay	Clay Till	Bedrock
γ	19.5 kN/m^3	21 kN/m^3	22 kN/m^3
C_u	60 kPa	125 kPa	250 kPa
ϕ	25^{O}	28^{O}	38^{O}
k_A	0.30	0.30	-
k_P	2.5	2.5	-

The load capacity for the governing design criteria as determined by these analysis are considered to be ultimate values. The factored ULS geotechnical capacity for horizontal loads on deep pile foundations should be determined by applying a GRF of 0.5. Specific cases can be analysed by ParklandGEO or alternate input parameters for various computer programs can be provided, upon request.

6.7 FROST DESIGN CONSIDERATIONS

Pile shafts will be subject to adfreezing stresses within the design depth of frost which is about 2.8 m at this site. Adfreezing pressures causing pile jacking should be assumed to average 80 kPa over the estimated depth of frost penetration. This adfreeze force is an ultimate load. In the case of driven steel friction piles, resistance to adfreeze uplift forces will be provided by the dead load acting on the pile, the weight of the pile and the frictional resistance of the shaft below the frost zone. The unfactored ultimate shaft friction values for the soil below the depth of frost given in Subsection 6.4.1 may be used to determine the required pile embedment to resist frost forces. The resisting forces should be 150 percent of the calculated adfreeze forces.

Frost heave forces will also act on the underside of pile caps and grade beams with upward heaving pressure in the order of 200 kPa or greater. The potential of frost heaving forces can be greatly reduced by the placement of a compressible material or by providing a void of at least 75 mm between the underside of the concrete cap or grade beam and soil. A product such as Voidform or an equivalent is recommended. If a compressible material be used as an alternative to the Voidform, the uplift pressure acting on the underside of the concrete may be taken as the crushing strength of the compressible medium. The finished grade adjacent to the foundation should be sloped away so the surface runoff is not allowed to infiltrate and collect in the void space or in the compressible medium. If water is allowed to accumulate in the void space or the compressible medium becomes saturated, the beneficial effect will be negated and frost heaving pressures will occur.

6.8 LATERAL EARTH PRESSURE

Lateral earth pressures will act against the marina perimeter. In addition, if a shored excavation is used, lateral earth pressures will act against the shoring walls. Three earth pressure cases will exist at this site.

1. Active Case. Active earth pressures (K_A) should be used behind retaining walls which are unrestrained at the top and flexible walls which are allowed to move away from the restrained soil mass (ie. shoring).

2. "At Rest" Case. "At rest" pressures (K_O) should be used behind retaining walls which are restrained at the top and would include typical basement walls. At-rest conditions should be assumed for any sections of the shoring wall required to support adjacent development to minimize potential loss of support for existing foundations.

3. Passive Case. Passive earth pressures (K_P) act on the front of a wall (ie. against the base of the wall). Horizontal stresses on the wall push against the soil creating a much larger resisting force than is produced by the active or at rest conditions.

Lateral earth pressures may be computed using the following equation:

$$P = K Q + K \gamma H$$

where:
- P = lateral earth pressure at depth H below ground level (kPa)
- Q = Any surcharge loading at the ground surface (kPa)
- K = coefficient of lateral earth pressure
- γ = total unit weight of backfill compacted to 95 % SPMDD (kN/m^3)
- H = depth below ground level

The preceding relationship makes no allowance for hydrostatic pressures to build up behind the wall. If groundwater is allowed to back up against the foundation walls, the following relationship may be used to calculate lateral pressures:

$$P = K_o Q + K_o (\gamma H + \gamma_w d) + \gamma_w$$

where: γ_w = unit weight of water (9.8 kN/m^3)
d = depth below design high water elevation (m)

Recommended design values for these parameters depend on the type of backfill used. Recommended design values are given in the following table.

TABLE 7
LATERAL EARTH PRESSURE PARAMETERS

Type of Backfill	Total Unit Weight (kN/m^3)	Coeff. of Lateral Earth Pressure		
		K_A	K_O	K_P
Native clay fill material	19.5	0.30	0.60	2.5
Native silt and sand fill material	20.5	0.35	0.50	2.5
Free draining granular material	21.0	0.33	0.45	3.0

The preceding relationship makes no allowance for additional horizontal forces due to frost to build up behind the wall on the assumption that frost protection will be utilized. The earth pressure relationship given above assumes nominal compaction of the backfill to a maximum of 95 percent SPMDD. Only light, hand operated equipment should be operated within 1.5 m of walls and walls should be braced prior to backfilling. If higher levels of compaction are proposed, the earth pressure relationship given above should be reviewed. If no frost protection is provided the active or at rest lateral earth pressures pushing on the wall should be increased by a factor of 2 for the depth of frost.

The soil conditions at the site are suitable to driven steel installations for shoring applications. Sheet pile wall, steel “H” piles with lagging and diaphragm wall are commonly used to support excavations. These walls can be further laterally supported by bracing struts or tie-back anchors to provide a lateral resistance to earth pressure from surrounding ground. The soil conditions at the site are best suited to sheet piles or steel “H” piles with lagging

6.9 CONCRETE

Water-soluble sulphate concentrations from the samples tested indicated negligible potential for chemical attack of subsurface concrete. Therefore, General Use (Type GU) hydraulic cement is suitable for use in all subsurface concrete in contact with native soil at the site in accordance with CSA Standard CAN3-A23.1-M04. The recommended minimum 28 day compressive strength is 25 MPa with a water cement ratio of 0.5. All concrete exposed to a freezing environment either during or after construction should be air entrained.

6.10 FLEXIBLE ASPHALT PAVEMENT

The proposed pavement design sections are based on the assumption that the pavement will be constructed on a stable, prepared subgrade with a California Bearing Ratio of 3.0. This is indicative of a relatively low level of subgrade support as expected during spring thaw when the subgrade soils will exist in a weakened condition. As previously discussed in Section 6.2, subgrade problems may be encountered depending on local weather and groundwater conditions at the time of construction. If soft subgrade conditions are encountered, it is assumed that the subgrade will be improved with coarse gravel to support construction traffic and paving activities.

Two flexible pavement designs are proposed for this site, one for light traffic in the parking areas; and one for the heavier traffic area on the access road and truck loading areas. The assumed loading for heavy truck traffic is 25 truck loadings per day. If it is anticipated that traffic will exceed these levels, the design section provided below should be reviewed.

TABLE 8
FLEXIBLE PAVEMENT DESIGN

	Light	Heavy
Asphalt Concrete	75 mm	100 mm
25 mm Crushed Base Gravel	150 mm	150 mm
Granular Sub-Base (minimum)	200 mm	300 mm

If it is proposed to reduce the ACP layer for the heavy section as cost savings it is suggested to increase the subbase thickness, because the cost of future overlay would be significantly less then repairing a subgrade problem. The pavement could be thickened in the future when the “serviceability performance” warrants an overlay. The thickness of subbase given above is considered to be the minimum requirement assuming no subgrade improvement is required. It is expected that areas of the parking lot will require subgrade improvements. Based on local experience the combined thickness of the gravel subbase and subgrade improvement gravel required to provide support for pavements may be up to 0.40 to 0.75 m thick.

The performance of the proposed pavement design sections will be, in part, dependent on achieving an adequate level of compaction in subgrade and pavement materials. The recommended levels of compaction for the granular materials in the pavement section should be a minimum of 98 percent of SPMDD. The asphalt concrete should be compacted to a minimum of 97 percent of Marshall density based on a 50 blow laboratory Marshall test. It is recommended to use pavement materials conforming to the following specifications:

TABLE 9
ASPHALT CONCRETE

Parameter	Specification
Stability (kN minimum)	8.0
Flow (mm)	2 - 4
Air Voids (percent)	3 - 5
VMA (minimum percent)	14.5
Asphalt Cement (penetration grade)	150 - 200 (A)

Aggregate materials for base and subbase gravel should be composed of sound, hard, durable particles free from organics and other foreign material. It is recommended to use aggregates conforming to the following City of Red Deer specifications.

TABLE 10
RECOMMENDED AGGREGATE SPECIFICATIONS

Material	City Specifications
Asphalt Gravel	Designation 5, Class c
Crushed Base Gravel	Designation 4, Class b
Subbase Gravel	Designation 3, Class b

A copy of the City of Red Deer aggregate specifications are provided in Appendix A. Based on availability of local materials at the time of tendering or construction, alternate materials could be considered upon review by the geotechnical engineer.

The parking areas should be sloped and graded to effectively remove all surface water as rapidly as possible. To minimize the occurrence of surface water ponding in the parking area, surface grades of at least 2 percent are recommended. Allowing water to pond on the pavement surface will lead to infiltration of the water into the subgrade which could result in weakening of the subgrade soils.

A geo-textile filter fabric is recommended as a separation barrier for all transitions between gravel and fine grained silty sand soils in areas of the parking lot which cannot be prepared as described in Section 6.2. Due to the shallow groundwater conditions and sand subgrade, filter cloth is recommended for all high and critical traffic areas of the site. In this application the filter cloth is used for separation not reinforcement, but it must be strong enough to withstand construction activities. For pavement applications it is recommended to use a non-woven filter fabric with a minimum Grab Tensile Strength of 900 N. The filter fabric should be provided with overlaps in conformance with the manufacturer's recommendations or at least 0.3 m, whichever is greater.

6.11 INSPECTION

It is recommended that on-site inspection and testing be performed to verify that actual site conditions are consistent with assumed conditions which meet or exceed design criteria. Based on the Alberta Building Code, adequate levels of inspection include: testing of engineered fill, review of all completed bearing surfaces for footings and full time inspection during construction deep foundations.

7.0 CLOSURE

This report is based on the findings at four deep borehole and three probe locations. If different subsoil and groundwater conditions are encountered, this office must be notified and recommendations submitted herein will be reviewed and revised as required. This report has been prepared for the exclusive use of the **Delta Land Company Inc.**, and their approved agents for the specified application to the proposed marina in SE 1-41-1-W5M, Lacombe County, Alberta. It has been prepared in accordance with generally accepted soil and foundation engineering practices. No other warranty, expressed or implied, is made. Use of the report is subject to acceptance of the General Terms and Conditions provided in Limitations of this report.

We trust this meets your present needs. If you have any question and comments regarding this information, please do not hesitate to contact our office.

Respectfully Submitted,
PARKLAND GEOTECHNICAL CONSULTING LTD.
A.P.E.G.G.A. Permit #07312

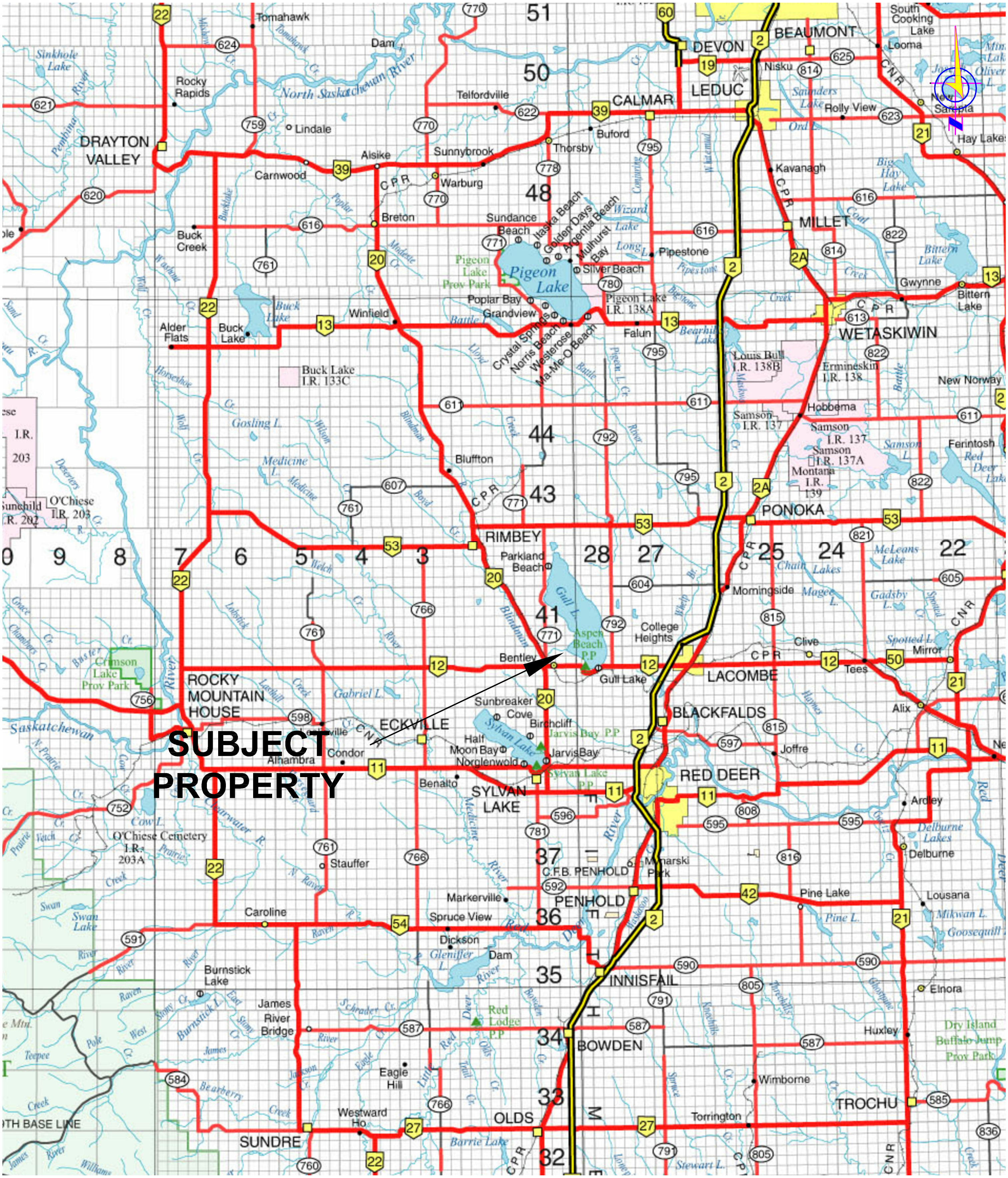
April 29, 2011

Phillip Auclair, E.I.T
Geo-Environmental Engineer

Mark Brotherton, P.Eng.
Principal Geotechnical Engineer

APPENDIX A

Figure 1 - Key Plan
Figure 2 - Site Plan
Figure 3 - Aerial Photograph
Logs (BH1 and BH7)
Soil Test Results
Aggregate Specifications
Explanation Sheets



Parkland GEO

CLIENT:

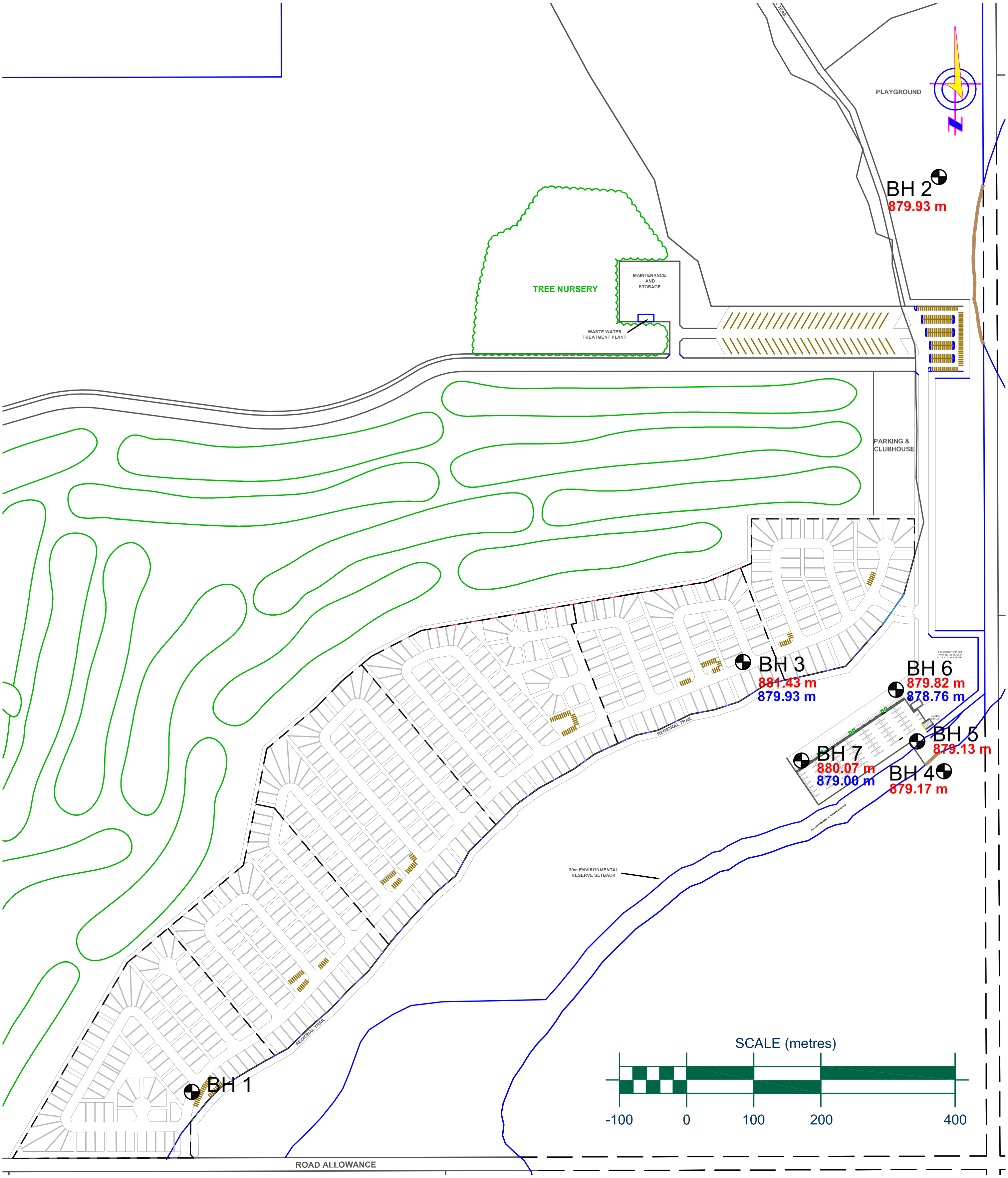
DELTA LAND COMPANY INC.

KEY PLAN

SANDY POINT DEVELOPMENT
SE1-41-1-W5M, LACOMBE COUNTY, AB

DRAWN:	CHK'D.:	REV #:	DATE:
PA	MDB	1	APRIL 2011

SCALE:	JOB NO.	DRAWING NO.
NTS	RD3773	FIGURE 1



CLIENT:

DELTA LAND COMPANY INC.

SITE PLAN

SANDY POINT DEVELOPMENT
SE 1-41-1-W5M, LACOMBE COUNTY, AB

DRAWN: PA
CHK'D.: MDB
REV #: 1
DATE: APRIL 2011
SCALE: 1:7500
JOB NO. RD3773
DRAWING NO. FIGURE 2

Parkland GEO	CLIENT: **DELTA LAND COMPANY INC.**	**2010 AERIAL PHOTOGRAPH**		
		SANDY POINT DEVELOPMENT SE 1-41-1-W5M, LACOMBE COUNTY, AB		
		DRAWN: PA	CHK'D.: MDB	REV #: 1
		DATE: APRIL 2011		
		SCALE: 1:10000	JOB NO. RD3773	DRAWING NO. FIGURE 3

ParklandGEO

BOREHOLE NO.: 01

CLIENT: Delta Land Company Inc.
SITE: Sandy Point Development
NOTES: Gull Lake, Lacombe County, Alberta

PROJECT NO.: RD3773
BH LOCATION:

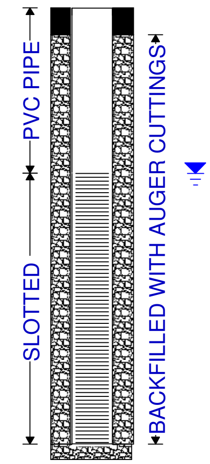
SUBSURFACE PROFILE

Depth (m)	Description	Moisture (Wp |-----X-----| Wl)	Type	Sample No	SPT (N)	Comments	Elevation (m)
0	GROUND SURFACE						0.00
0–5.0	***Till*** Clay, silty, some sand, trace gravel, stiff, low plastic, brown, occasional rust stains, occasional coal inclusions, moist. - Frost to 0.4m.						-0.40
1.0		21					
1.0–1.5			G	1G1			
1.5–2.0				1D1	11		
2.0		22					
2.5		23				GRAIN SIZE ANALYSIS: CLAY=30% SILT=37% SAND=32% GRAVEL=1%	
2.5–3.0			G	1G2			
3.0		24					
3.5–4.0			G	1G3			
4.0		22					
4.5–5.0				1G4	17		
5.0	End of hole at 5.0m. 25mm PVC standpipe installed. Backfilled with auger cuttings and Bentonite cap. Dry upon completion. Water at 1.83m on March 23, 2011.	21					-5.00

Moisture scale: 25, 50, 75

Depth scale (m): 0, 1, 2, 3, 4, 5, 6, 7, 8, 9, 10, 11, 12, 13, 14, 15

Well Completion Details



LOGGED BY: SS	GROUND ELEVATION:
CONTRACTOR: All Service Drilling Inc.	NORTHING:
RIG/METHOD: Truck Mount / Solid Stem Auger	EASTING:
DATE: March 14, 2011	
CALIBRATION:	

ParklandGEO

CLIENT: Delta Land Company Inc.
SITE: Sandy Point Development
NOTES: Gull Lake, Lacombe County, Alberta

BOREHOLE NO.: 02

PROJECT NO.: RD3773
BH LOCATION:

SUBSURFACE PROFILE

Depth (m)	Description	Symbol	Moisture (Wp |-----X-----| Wl) 25 50 75	Type	Sample No	SPT (N)	Comments	Well Completion Details	Elevation (m)
	GROUND SURFACE								879.93
0	***Sand*** Trace silt, medium grained, poorly graded, compact, brown, moist.								879.03
1	- Saturated at 0.9m.		25	G	2G1				878.43
	- Grey at 1.5m.								
2	***Sand and Silt*** Fine grained, poorly graded, compact, grey, saturated.		28						877.83
									877.43
3	- Clay from 2.5 to 2.6m.		43	G	2G2				876.93
	End of hole at 3.0m. Backfilled with auger cuttings. Wet upon completion.								

Depth scale: 0, 1, 2, 3, 4, 5, 6, 7, 8, 9, 10, 11, 12, 13, 14, 15

LOGGED BY: SS
CONTRACTOR: All Service Drilling Inc.
RIG/METHOD: Truck Mount / Solid Stem Auger
DATE: March 14, 2011
CALIBRATION:

GROUND ELEVATION: 879.934 m
NORTHING: 5821673 m
EASTING: 703437 m

BOREHOLE NO.: 03

CLIENT: Delta Land Company Inc.
SITE: Sandy Point Development
NOTES: Gull Lake, Lacombe County, Alberta

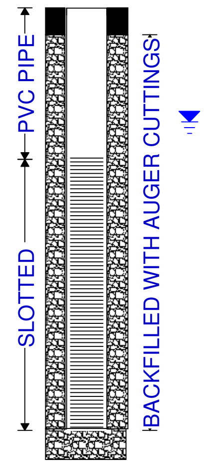
PROJECT NO.: RD3773
BH LOCATION:

SUBSURFACE PROFILE

Depth (m)	Description	Moisture (Wp |-----X-----| Wl) 25 50 75	Type	Sample No	SPT (N)	Comments	Elevation (m)
	GROUND SURFACE						881.43
0–0.95	***Sand*** Trace silt, fine grained, poorly graded, compact, brown, damp.						
1		23	G	3G1			
0.95–5.0	***Till*** Clay, silty, little sand, trace gravel, stiff, low plastic, brown, occasional rust stains, occasional coal inclusions, moist.						
2		19		3D1	9		
3	- Sand lense (1mm).	20	G	3G2			878.63
4		20					
5		17		3D2	64	50 Blows for 150mm.	876.43

End of hole at 5.0m.
25mm PVC standpipe installed.
Backfilled with auger cuttings and Bentonite cap.
Water at 4.2m upon completion.
Water at 1.26m on March 23, 2011.

Well Completion Details



LOGGED BY: SS	GROUND ELEVATION: 881.428 m
CONTRACTOR: All Service Drilling Inc.	NORTHING: 5820951 m
RIG/METHOD: Truck Mount / Solid Stem Auger	EASTING: 703145 m
DATE: March 14, 2011	
CALIBRATION:	

BOREHOLE NO.: 04

CLIENT: Delta Land Company Inc.
SITE: Sandy Point Development
NOTES: Gull Lake, Lacombe County, Alberta

PROJECT NO.: RD3773
BH LOCATION:

SUBSURFACE PROFILE

Depth (m)	Description	Symbol	Moisture (Wp |-----X-----| Wl) 25 50 75	Type	Sample No	SPT (N)	Comments	Well Completion Details	Elevation (m)
0	GROUND SURFACE								879.16
	Ice								
	Organic Silt Trace sand, soft, low plastic, grey, saturated. - Frost to 1.5m.			G	4G1		GRAIN SIZE ANALYSIS: CLAY=22% SILT=56% SAND=22% GRAVEL=0% ORGANIC=8.0%		
1			21						
									877.66
2	***Till*** Clay, silty, trace sand, trace gravel, stiff, medium plastic, brown, occasional rust stains, occasional coal inclusions, saturated.		30						
				G	4G2				
3			40						876.16
	End of hole at 3.0m. Blackfilled with auger cuttings. Dry upon completion.								

LOGGED BY: SS
CONTRACTOR: All Service Drilling Inc.
RIG/METHOD: Truck Mount / Solid Stem Auger
DATE: March 14, 2011
CALIBRATION:

GROUND ELEVATION: 879.165 m
NORTHING: 5820790 m
EASTING: 703444 m

BOREHOLE NO.: 05

CLIENT: Delta Land Company Inc.
SITE: Sandy Point Development
NOTES: Gull Lake, Lacombe County, Alberta

PROJECT NO.: RD3773
BH LOCATION:

SUBSURFACE PROFILE

Depth (m)	Description	Symbol	Moisture (Wp |-----X-----| Wl) 25 50 75	Type	Sample No	SPT (N)	Comments	Well Completion Details	Elevation (m)
0	GROUND SURFACE								879.13
	Sand and Silt Fine grained, poorly graded, compact, brown, wet.								878.83
1	***Till*** Clay, silty, trace sand, trace gravel, stiff, low plastic, brown, occasional rust stains, occasional coal, saturated		22	G	5G1				877.63
2			18						
3	***Till*** Sand, silty, little clay, trace gravel, fine grained, poorly graded, compact, brown, moist.		21	G	5G2				876.13
4	End of hole at 3.0m. Backfilled with auger cuttings. Water at 2.7m upon completion.								
5									
6									
7									
8									
9									
10									
11									
12									
13									
14									
15									

LOGGED BY: SS
CONTRACTOR: All Service Drilling Inc.
RIG/METHOD: Truck Mount / Solid Stem Auger
DATE: March 14, 2011
CALIBRATION:

GROUND ELEVATION: 879.126 m
NORTHING: 5820829 m
EASTING: 703411 m

BOREHOLE NO.: 06

CLIENT: Delta Land Company Inc.
SITE: Sandy Point Development
NOTES: Gull Lake, Lacombe County, Alberta

PROJECT NO.: RD3773
BH LOCATION:

SUBSURFACE PROFILE

Depth (m)	Description	Moisture (Wp |-----X-----| Wl)	Type	Sample No	SPT (N)	Comments	Elevation (m)
0	GROUND SURFACE						879.82
0–0.3	**Sand** Silty, fine grained, poorly graded, compact, brown, moist.		G	6G1			879.52
0.3–1.0	**Organic Soil** Firm, black, moist.	41	G	6G2			
1.0–2.5	**Clay** Silty, trace sand, very stiff, medium plastic, brown, occasional rust stains, moist.	37	G / D	6G3 / 6D1	12		877.32
2.5–4.6	**Till** Clay, silty, trace sand, trace gravel, low plastic, stiff to very stiff, brown, occasional rust stains, occasional coal inclusions, wet.	20, 20	G / G	6G4 / 6G5			875.22
4.6–6.0	**Weathered Bedrock** Siltstone, very stiff, non-plastic, grey, moist.	21, 21	D / G	6D2 / 6G6	40		873.82
6.0–7.5	**Weathered Bedrock** Sandstone, very dense, grey, moist.	21	G	6G7			872.32
7.5–10.5	**Weathered Bedrock** Siltstone, very stiff, non-plastic, grey, moist.	26, 29, 26	D / G	6D3 / 6G8	50	50 Blows for 125mm.	869.32

End of hole at 10.5m.
25mm PVC standpipe installed.
Backfilled with auger cuttings and Bentonite cap.
Water at 3.0m upon completion.
Water at 1.06m on March 23, 2011.

Well Completion Details: PVC PIPE; SLOTTED; BACKFILLED WITH AUGER CUTTINGS

LOGGED BY: SS
CONTRACTOR: All Service Drilling Inc.
RIG/METHOD: Truck Mount / Solid Stem Auger
DATE: March 14, 2011
CALIBRATION:

GROUND ELEVATION: 879.816 m
NORTHING: 5820799 m
EASTING: 703232 m

ParklandGEO

CLIENT: Delta Land Company Inc.
SITE: Sandy Point Development
NOTES: Gull Lake, Lacombe County, Alberta

BOREHOLE NO.: 07
PROJECT NO.: RD3773
BH LOCATION:

SUBSURFACE PROFILE

Depth (m)	Description	Moisture (Wp |-----X-----| Wl)	Elevation (m)
0	GROUND SURFACE		880.07
0–4.4	***Till*** Clay, silty, trace sand, trace gravel, stiff to very stiff, low plastic, brown, occasional rust stains, occasional coal inclusions, wet.	22, 20, 20, 20, 21	875.67
4.4–8.4	***Weathered Bedrock*** Siltstone, very stiff, non-plastic, grey, moist.	21, 20, 24, 18	871.67
8.4–10.5	***Weathered Bedrock*** Sandstone, very dense, grey, moist.	25, 26	869.57

Type	Sample No	SPT (N)	Comments
	B1		
G	7G1		GRAIN SIZE ANALYSIS: CLAY=27% SILT=37% SAND=34% GRAVEL=2%
	7D1	23	
G	7G2		
	7D2	50	50 Blows for 25mm.
G	7G3		
G	7G4		
	7D3	50	50 Blows for 50mm.

Well Completion Details: PVC PIPE; SLOTTED; BACKFILLED WITH AUGER CUTTINGS

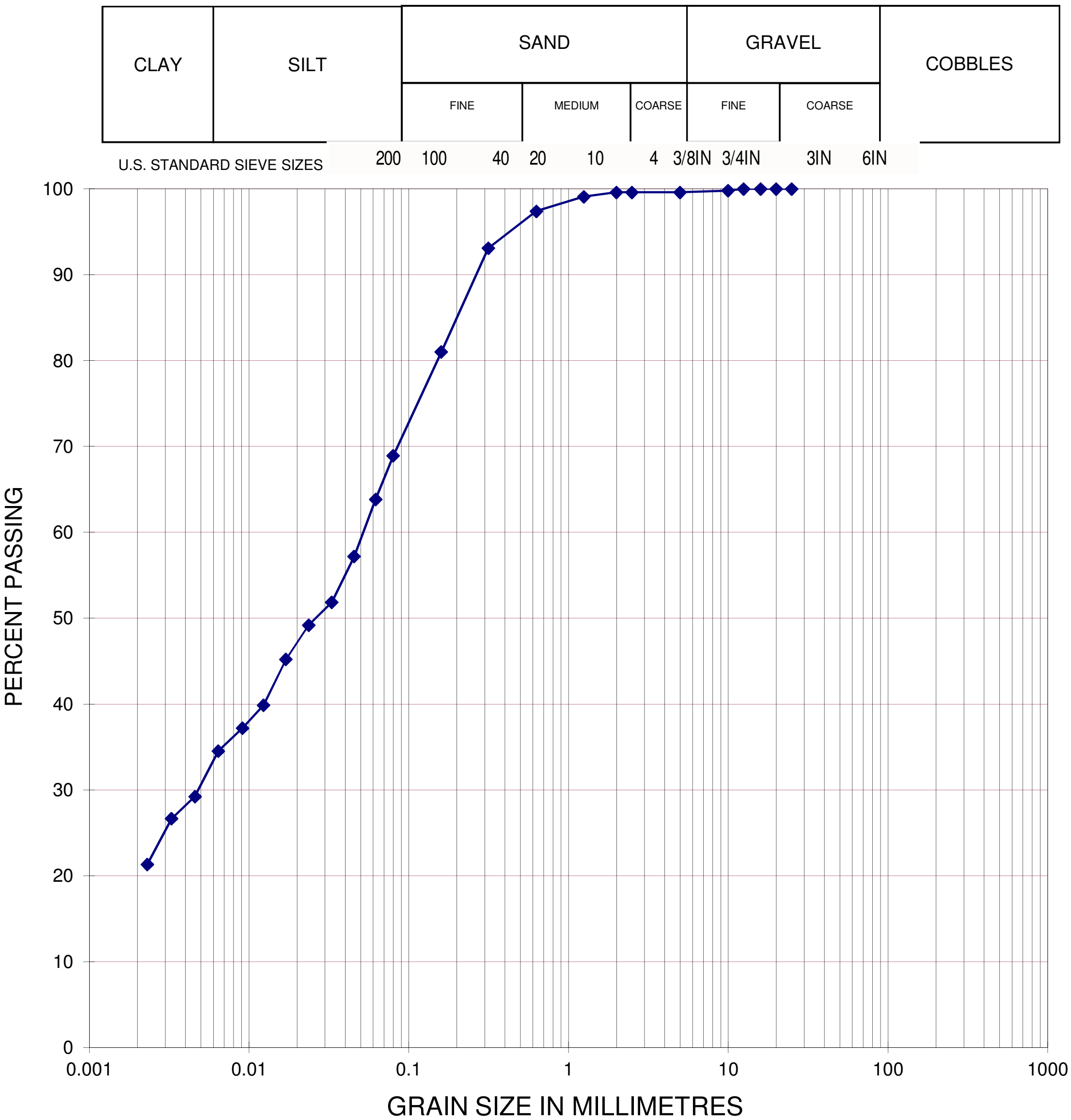
End of hole at 10.5m.
25mm PVC standpipe installed.
Backfilled with auger cuttings and Bentonite cap.
Water at 3.0m upon completion.
Water at 1.07m on March 23, 2011.

LOGGED BY: SS
CONTRACTOR: All Service Drilling Inc.
RIG/METHOD: Truck Mount / Solid Stem Auger
DATE: March 14, 2011
CALIBRATION:

GROUND ELEVATION: 880.065 m
NORTHING: 5820888 m
EASTING: 703361 m

PROJECT	Sandy Point Development		
PROJECT #	RD3773		
BOREHOLE	1	**DATE**	Mar 30/11
DEPTH	2.5m	**TECH**	RM
SAMPLE	1G2		
LOCATION			

GRAIN SIZE DISTRIBUTION



COMMENTS:

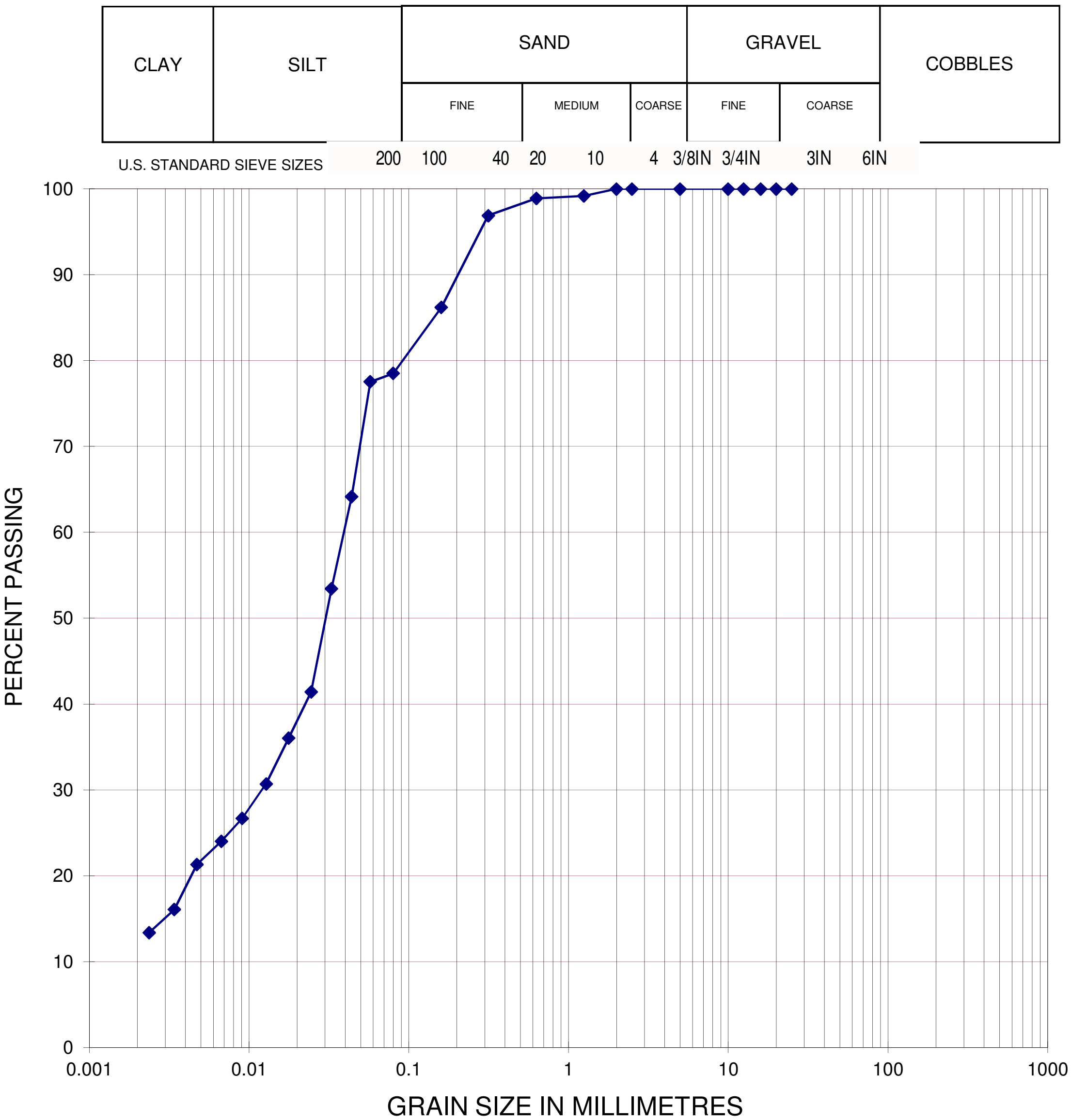
% Retained on 2 mm seive
Soil Type: Silt, Some Sand, Some Clay

SUMMARY

D10 =		GRAVEL	0.40%
D30 =		SAND	32.12%
D60 =		SILT	37.08%
CU =		CLAY	30.39%
CC =			

PROJECT	Sandy Point Development		
PROJECT #	RD3773		
BOREHOLE	4	**DATE**	Mar 30/11
DEPTH	0.3m	**TECH**	RM
SAMPLE	4G2		
LOCATION			

GRAIN SIZE DISTRIBUTION



COMMENTS:

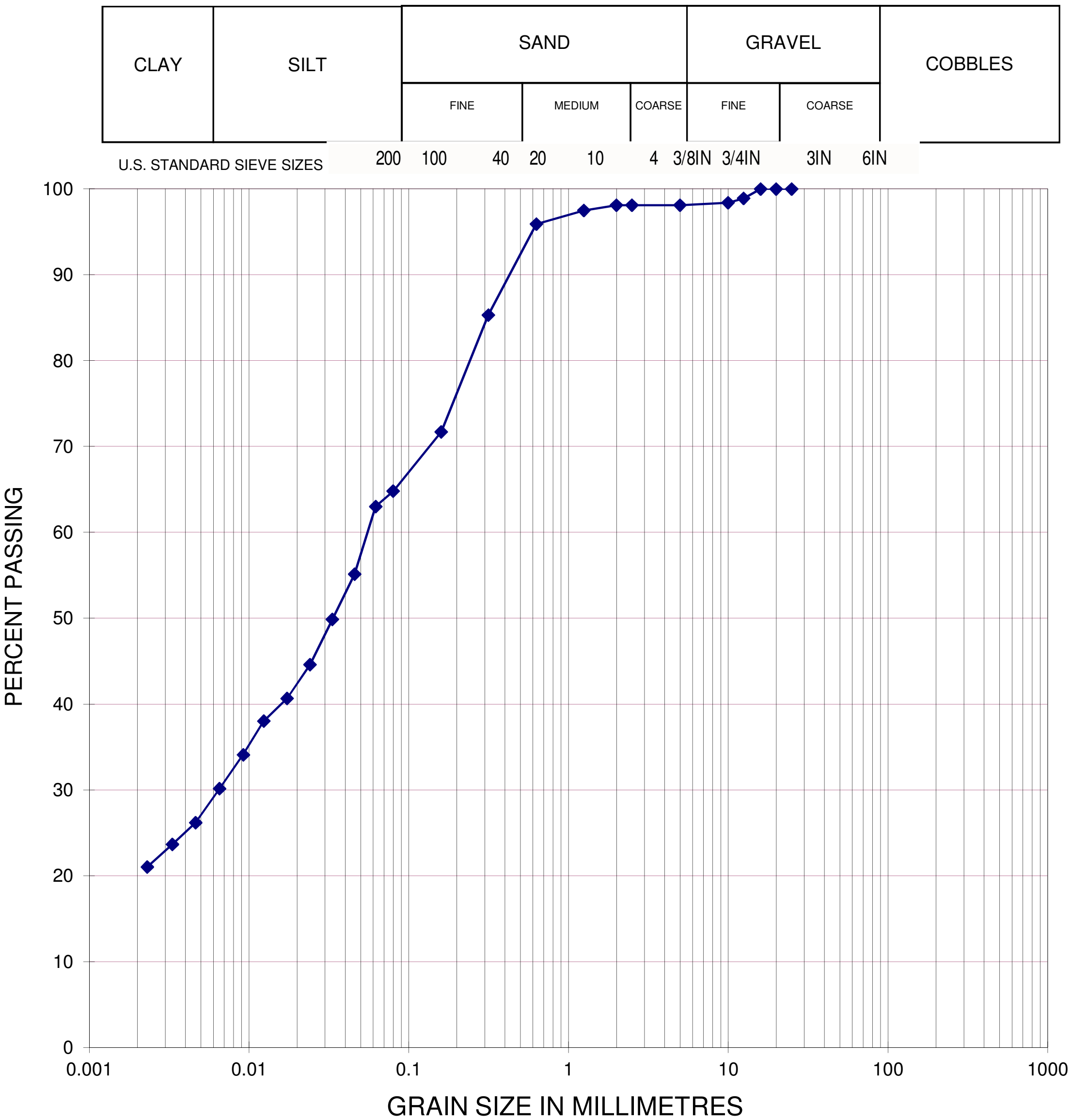
% Retained on 2 mm seive
Soil Type: Silt, Some Sand, Some Clay

SUMMARY

D10 =		GRAVEL	0.00%
D30 =		SAND	21.71%
D60 =		SILT	56.60%
CU =		CLAY	21.69%
CC =			

PROJECT	Sandy Point Development		
PROJECT #	RD3773		
BOREHOLE	7	**DATE**	Mar 30/11
DEPTH	2.3m	**TECH**	RM
SAMPLE	7G1		
LOCATION			

GRAIN SIZE DISTRIBUTION



COMMENTS:

% Retained on 2 mm seive
Soil Type: Silt, Some Sand, Some Clay

SUMMARY

D10	=	GRAVEL	1.90%
D30	=	SAND	33.81%
D60	=	SILT	37.34%
CU	=	CLAY	26.95%
CC	=		

PROJECT#	Sandy Point Development
PROJECT	RD3773
BOREHOLE	1
DEPTH	2.5m
SAMPLE #	1G2
DATE	Mar 30/11
TECH	RM

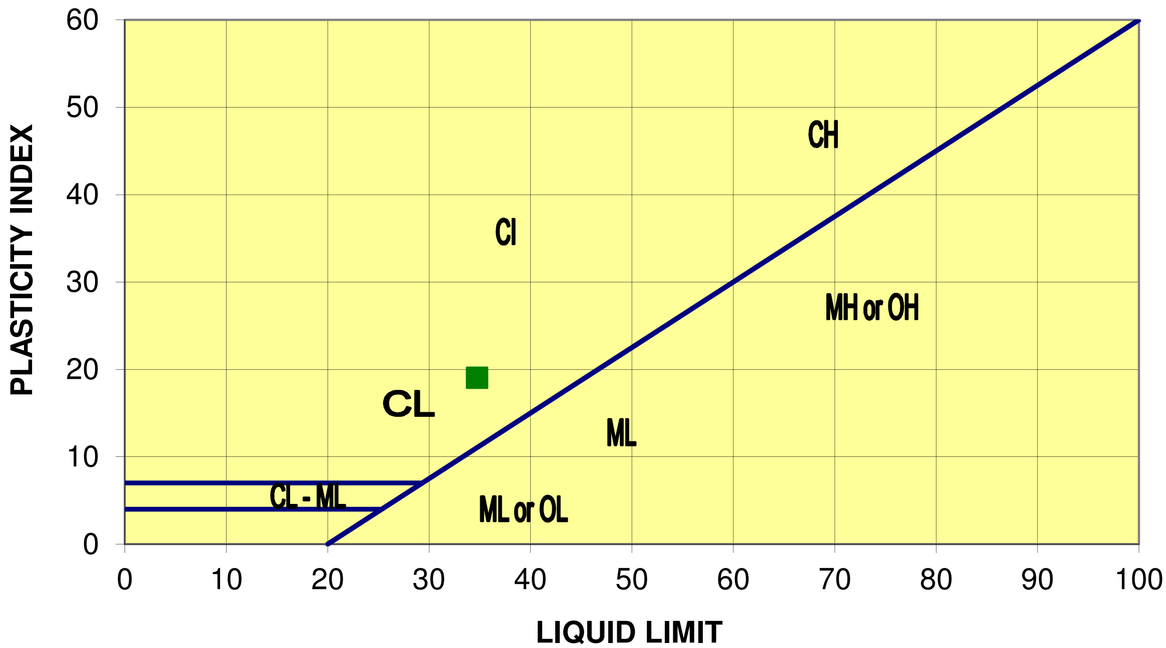
SOIL PLASTICITY SUMMARY

LIQUID LIMIT (LL)		
Trial No.	1	2
No. Blows	23	24
Wt. Sample Wet + Tare	36.066	40.242
Wt. Sample Dry + Tare	30.932	33.962
Wt. Water	5.134	6.280
Tare Container	16.257	16.041
Wt. Dry Soil	14.675	17.921
Moisture Content	34.985	35.043
Corrected for Blow Count	34.633	34.870
Liquid Limit Average	**34.8**	

PLASTIC LIMIT (PL)			
Trial No.	1	2	3
Wt. Wet Worm + Tare	9.379	8.897	9.238
Wt. Dry Worm + Tare	8.959	8.539	8.846
Wt. Water	0.420	0.358	0.392
Tare Container	6.301	6.259	6.339
Wt. Dry Worm	2.658	2.280	2.507
Moisture Content	15.801	15.702	15.636
Plastic Limit Average		**15.7**	

Table 2 - 1 5

PLASTICITY INDEX (PI) = LL-PL **19.0**



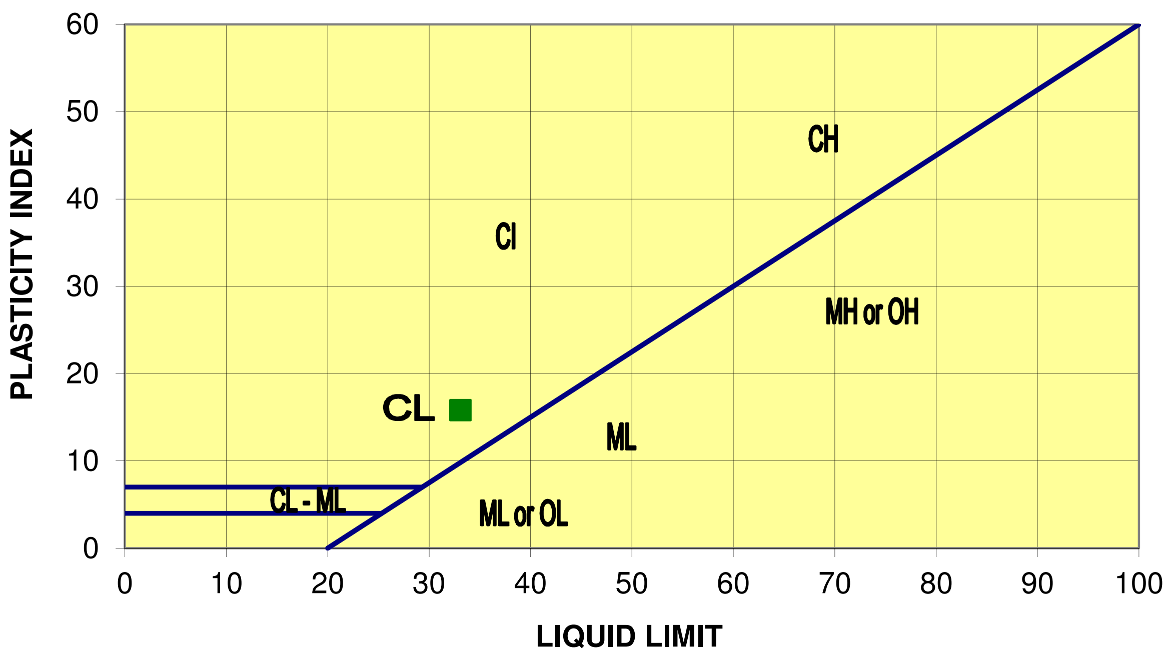
PROJECT#	Sandy Point Development
PROJECT	Rd3773
BOREHOLE	7
DEPTH	2.3m
SAMPLE #	7G1
DATE	Mar 30/11
TECH	RM

SOIL PLASTICITY SUMMARY

LIQUID LIMIT (LL)		
Trial No.	1	2
No. Blows	21	22
Wt. Sample Wet + Tare	38.688	43.960
Wt. Sample Dry + Tare	33.049	36.948
Wt. Water	5.639	7.012
Tare Container	16.265	16.209
Wt. Dry Soil	16.784	20.739
Moisture Content	33.597	33.811
Corrected for Blow Count	32.896	33.292
Liquid Limit Average	**33.1**	

PLASTIC LIMIT (PL)			
Trial No.	1	2	3
Wt. Wet Worm + Tare	9.418	9.486	8.849
Wt. Dry Worm + Tare	8.959	9.025	8.471
Wt. Water	0.459	0.461	0.378
Tare Container	6.305	6.351	6.275
Wt. Dry Worm	2.654	2.674	2.196
Moisture Content	17.295	17.240	17.213
Plastic Limit Average		**17.2**	

PLASTICITY INDEX (PI) = LL-PL **15.8**



2910 12TH STREET NE
CALGARY, ALBERTA
CANADA T2E 7P7
TEL (403)735-2005
FAX (403)735-2771
http://www.agatlabs.com

CLIENT NAME: **PARKLAND GEOTECHNICAL CONSULTING**
102, 4756 RIVERSIDE DRIVE
RED DEER, AB T4N2N7

ATTENTION TO: **Phillip Auclair**

PROJECT NO: **RD3773**

AGAT WORK ORDER: **11R483858**

SOIL ANALYSIS REVIEWED BY: **Loan Nguyen, Analyst**

DATE REPORTED: **Apr 08, 2011**

PAGES (INCLUDING COVER): **4**

VERSION*: **1**

Should you require any information regarding this analysis please contact your client services representative at (403) 735-2005, or at 1-866-764-7554

*NOTES

All samples will be disposed of within 30 days following analysis. Please contact the lab if you require additional sample storage time.

Member of: Association of Professional Engineers, Geologists and Geophysicists of Alberta (APEGGA)
Western Enviro-Agricultural Laboratory Association (WEALA)
Environmental Services Association of Alberta (ESAA)

AGAT Laboratories is accredited to ISO/IEC 17025 by the Canadian Association for Laboratory Accreditation Inc. (CALA) and/or Standards Council of Canada (SCC) for specific tests listed on the scope of accreditation. AGAT Laboratories (Mississauga) is also accredited by the Canadian Association for Laboratory Accreditation Inc. (CALA) for specific drinking water tests. Accreditations are location and parameter specific. A complete listing of parameters for each location is available from www.cala.ca and/or www.scc.ca. The tests in this report may not necessarily be included in the scope of accreditation.

Results relate only to the items tested

Certificate of Analysis

AGAT WORK ORDER: 11R483858

PROJECT NO: RD3773

AGAT Laboratories
2910 12TH STREET NE
CALGARY, ALBERTA
CANADA T2E 7P7
TEL (403)735-2005
FAX (403)735-2771
http://www.agatlabs.com

CLIENT NAME: PARKLAND GEOTECHNICAL CONSULTING

ATTENTION TO: Phillip Auclair

Soil Analysis - Sulfate

DATE SAMPLED: Mar 14, 2011 | **DATE RECEIVED: Apr 06, 2011** | **DATE REPORTED: Apr 08, 2011** | **SAMPLE TYPE: Soil**

Parameter	Unit	G / S	RDL	MC6-2m 2338805	MC7-2m 2338808
Sulfate, Soluble	mg/L		2	70	63

Comments: RDL - Reported Detection Limit; G / S - Guideline / Standard

Certified By:

2910 12TH STREET NE
CALGARY, ALBERTA
CANADA T2E 7P7
TEL (403)735-2005
FAX (403)735-2771
http://www.agatlabs.com

Quality Assurance

CLIENT NAME: PARKLAND GEOTECHNICAL CONSULTING **AGAT WORK ORDER: 11R483858**

PROJECT NO: RD3773 **ATTENTION TO: Phillip Auclair**

Soil Analysis

RPT Date: Apr 08, 2011			DUPLICATE			Method Blank	REFERENCE MATERIAL			METHOD BLANK SPIKE			MATRIX SPIKE		
PARAMETER	Batch	Sample Id	Dup #1	Dup #2	RPD		Measured Value	Acceptable Limits		Recovery	Acceptable Limits		Recovery	Acceptable Limits	
								Lower	Upper		Lower	Upper		Lower	Upper
Soil Analysis - Sulfate															
Sulfate, Soluble	154	7748	29	28	4.5%	< 2	96%	90%	110%		90%	110%	97%	90%	110%

Certified By:

AGAT Laboratories is accredited to ISO/IEC 17025 by the Canadian Association for Laboratory Accreditation Inc. (CALA) and/or Standards Council of Canada (SCC) for specific tests listed on the scope of accreditation. AGAT Laboratories (Mississauga) is also accredited by the Canadian Association for Laboratory Accreditation Inc. (CALA) for specific drinking water tests. Accreditations are location and parameter specific. A complete listing of parameters for each location is available from www.cala.ca and/or www.scc.ca. The tests in this report may not necessarily be included in the scope of accreditation.

Results relate only to the items tested

2910 12TH STREET NE
CALGARY, ALBERTA
CANADA T2E 7P7
TEL (403)735-2005
FAX (403)735-2771
http://www.agatlabs.com

Method Summary

CLIENT NAME: PARKLAND GEOTECHNICAL CONSULTING **AGAT WORK ORDER: 11R483858**

PROJECT NO: RD3773 **ATTENTION TO: Phillip Auclair**

PARAMETER	AGAT S.O.P	LITERATURE REFERENCE	ANALYTICAL TECHNIQUE
Soil Analysis			
Sulfate, Soluble	SOIL 0110; SOIL 0120; INST 0140	SHEPPARD 2007; EATON 2005	ICP/OES

Results relate only to the items tested

DESIGNATION		1			2					3				4			5	6		7	8
CLASS (mm)		10	12.5	16	16	20	25	40	50	12.5A	12.5B	12.5C	16	20	25	40	10	80	125	40	40
PER CENT PASSING METRIC SIEVE (CGSB 8 - GP - 2M) µm	125 000																		100		
	80 000																	100			
	50 000								100									55-100	55-100		
	40 000							100								100				100	100
	25 000						100		63-90						100			38-100	38-100		
	20 000					100								100		55-90					
	16 000			100	100		70-94	55-85	47-79				100					32-85	32-85		
	12 500		100	80-92						100	100	100	72-95								
	10 000	100	83-92	70-84	70-93	63-86	52-79	44-74	38-70	35-65	55-75	70-93	53-85	35-77	30-77	25-72	100			85-100	78-95
	5 000	60-75	55-70	50-65	50-70	40-67	35-64	32-62	28-59	0-15	0-15	30-70	27-64	15-55	15-55	8-55	45-70	20-65	20-65		60-85
	1250	30-45	30-45	30-45	26-45	20-43	18-43	17-43	16-42	0-3	0-3	9-34	9-34	0-30	0-30	0-30	20-45			40-100	27-57
	630	22-38	22-38	22-38	19-38	14-34	12-34	12-34	12-34												
	315	15-30	15-30	15-30	14-30	9-26	8-26	8-26	8-26			0-18	0-18				9-22	6-30	6-30	17-100	5-29
	160	9-20	9-20	9-20	9-20	5-18	5-18	5-18	5-18			0-13	0-13				5-15				0-15
	80	4-10	4-10	4-10	4-10	2-10	2-10	2-10	2-10	0-2	0-2	0-8	0-8	0-12	0-12	0-12	0-10	2-10	2-15	6-30	0-5
% FRACTURE BY WEIGHT (2 FACES)	ALL +5000	60+	60+	60+	60+	60+	60+	50+	40+	75+	75+	60+	60+	40+	40+	25+	N/A	N/A	N/A	N/A	N/A
PLASTICITY INDEX (PI)		0-4	0-4	0-4	0-6	0-6	0-6	0-6	0-6	N/A	N/A	0-4	0-4	0-8	0-8	0-8	0-6	0-8	0-8	0-10	0-5
L.A. ABRASION LOSS PER CENT MAX.		40	40	40	50	50	50	50	50	35	35	35	35	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
FLAKINESS INDEX		N/A								MAX 15		N/A									
COEFFICIENT OF UNIFORMITY (Cu)		N/A																		3+	N/A

DESIGNATION

1. ASPHALT CONCRETE AGGREGATE (CLASS 10 FOR SURFACE PREPARATION COURSE ONLY)
2. GRANULAR AND ASPHALT STABILIZED BASE COURSES, SUB-BASES AND DUST ABATEMENT AGGREGATES.
3. SEAL COAT AGGREGATE
4. GRAVEL SURFACING AGGREGATE
5. SANDING MATERIAL
6. PIT-RUN GRAVEL FILL
7. CEMENT STABILIZED BASE COURSE AGGREGATE
8. GRANULAR FILTER AGGREGATE

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Alberta
TRANSPORTATION AND UTILITIES

CHART	3.2 A
	Date
Original	MARCH 1984
Revised	DEC. 1985
Revised	FEB. 1987
Revised	MAR. 1988

SPECIFICATIONS FOR AGGREGATE

EXPLANATION OF TERMS AND SYMBOLS

The terms and symbols used on the borehole logs to summarize the results of the field investigation and subsequent laboratory testing are described on the following two pages.

The borehole logs are a graphical representation summarizing the soil profile as determined during site specific field investigation. The borehole logs may include test data from laboratory soil testing, if applicable. The materials, boundaries and conditions have been established only at the borehole locations at the time of drilling. The soil conditions shown on the borehole logs are not necessarily representative of the subsurface conditions elsewhere across the site. The transitions in soil profile usually have gradual rather than distinct unit boundaries as shown on this graphical representation.

1. **PRINCIPAL SOIL TYPE** - The major soil type by weight of material or by behavior.

Material	Grain Size
Boulders	Larger than 300 mm
Cobbles	75 mm to 300 mm
Coarse Gravel	19 mm to 75 mm
Fine Gravel	5 mm to 19 mm
Coarse Sand	2 mm to 5 mm
Medium Sand	0.425 mm to 2 mm
Fine Sand	0.75 mm to 0.425 mm
Silt & Clay	Smaller than 0.075 mm

2. **DESCRIPTION OF MINOR SOIL TYPE** - Minor soil types are identified by weight of minor component.

Percent	Descriptor
35 to 50	and
20 to 35	some
10 to 20	little
1 to 10	trace

3. **RELATIVE STRENGTH OF COARSE GRAINED SOIL** - The following terms are used relative to Standard Penetration Test (SPT), ASTM D1586, N value for blows per 300 mm.

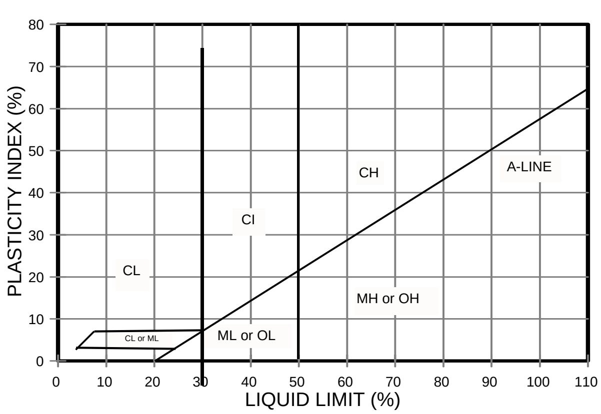
Description	N Value
Very Loose	Less than 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very Dense	Over 50

4. **CONSISTENCY OF FINED GRAINED SOIL** - The following terms are used relative to unconfined strength in kPa and Standard Penetration Test (SPT), ASTM D1586, N value for blows per 300 mm.

Description	Unconfined Compressive Strength (kPa)	N Value
Very Soft	less than 25	Less than 2
Soft	25 to 50	2 to 4
Firm	50 to 100	4 to 8
Stiff	100 to 200	8 to 15
Very Stiff	200 to 380	15 to 30
Hard	Over 380	Over 30

MODIFIED UNIFIED CLASSIFICATION SYSTEM FOR SOILS

MAJOR DIVISION			GROUP SYMBOL	GRAPH SYMBOL	TYPICAL DESCRIPTION	LABORATORY CLASSIFICATION CRITERIA	
COARSE GRAINED SOILS (MORE THAN HALF BY WEIGHT LARGER THAN NO. 200 SIEVE)	GRAVELS MORE THAN HALF COARSE GRAINS LARGER THAN NO. 4 SIEVE	CLEAN GRAVELS (LITTLE OR NO FINES)	GW		WELL GRADED GRAVELS, LITTLE OR NO FINES	$C_U = \frac{D_{60}}{D_{10}} > C_C = \frac{(D_{30})^2}{D_{10} \times D_{60}} = 1 \text{ to } 3$	
			GP		POORLY GRADED GRAVELS, GRAVEL-SAND MIXTURES WITH LITTLE OR NO	NOT MEETING ALL OF THE ABOVE REQUIREMENTS	
		DIRTY GRAVELS (WITH SOME FINES)	GM		SILTY GRAVELS, GRAVEL-SAND-SILT MIXTURES	CONTENT OF FINES EXCEEDS 12 %	ATTERBERG LIMITS BELOW "A" LINE OR P.I. LESS THAN 4
			GC		CLAYEY GRAVELS, GRAVEL-SAND-CLAY MIXTURES		ATTERBERG LIMITS ABOVE "A" LINE OR P.I. MORE THAN
	SANDS MORE THAN HALF FINE GRAINS SMALLER THAN NO. 4 SIEVE	CLEAN SANDS (LITTLE OR NO FINES)	SW		WELL GRADED SANDS, GRAVELLY SANDS WITH LITTLE OR NO FINES	$C_U = \frac{D_{60}}{D_{10}} > C_C = \frac{(D_{30})^2}{D_{10} \times D_{60}} = 1 \text{ to } 3$	
			SP		POORLY GRADED SANDS, LITTLE OR NO FINES	NOT MEETING ALL OF THE ABOVE REQUIREMENTS	
		DIRTY SANDS (WITH SOME FINES)	SM		SILTY SANDS, SAND-SILT MIXTURES	CONTENT OF FINES EXCEEDS 12 %	ATTERBERG LIMITS BELOW "A" LINE OR P.I. LESS THAN 4
			SC		CLAYEY SANDS, SAND-CLAY MIXTURES		ATTERBERG LIMITS ABOVE "A" LINE OR P.I. MORE THAN
FINE-GRAINED SOILS (MORE THAN HALF BY WEIGHT PASSES NO. 200 SIEVE)	SILTS BELOW "A" LINE NEGLIGIBLE ORGANIC CONTENT	$W_L < 50\%$	ML		INORGANIC SILTS & VERY FINE SANDS, ROCK FLUOR, SILTY SANDS OF SLIGHT	CLASSIFICATION IS BASED ON THE PLASTICITY CHART BELOW	
		$W_L > 50\%$	MH		INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS, FINE SANDY OR SILTY		
	CLAYS ABOVE "A" LINE ON PLASTICITY CHART NEGLIGIBLE ORGANIC CONTENT	$W_L < 30\%$	CL		INORGANIC CLAYS OF LOW PLASTICITY, GRAVELLY, SANDY OR		
		$30\% < W_L < 50\%$	CI		INORGANIC CLAYS OF MEDIUM PLASTICITY, SILTY CLAYS		
		$W_L > 50\%$	CH		INORGANIC CLAYS OF HIGH PLASTICITY		
	ORGANIC SILTS & CLAYS BELOW "A" LINE ON CHART	$W_L < 50\%$	OL		ORGANIC SILT, AND ORGANIC SILTY CLAYS OF LOW PLASTICITY		
		$W_L > 50\%$	OH		ORGANIC CLAYS OF HIGH PLASTICITY		
HIGHLY ORGANIC SOILS			Pt		PEAT AND OTHER HIGHLY ORGANIC SOILS	STRONG COLOR OR ODOR, AND OFTEN FIBROUS TEXTURE	



NOTES ON SOIL CLASSIFICATION AND DESCRIPTION:

5. Soils are classified and described according to their engineering properties and behaviour.
6. Boundary classifications for soils with characteristics of two groups are given combined group symbols, eg. GW-GC is a well graded gravel-sand mixture with clay binder between 5 and 12 %.
7. Soil classification is in accordance with the Unified Soil Classification System, with the exception that an inorganic clay of medium plasticity (CI) is recognized.
8. The use of modifying adjectives may be employed to define the estimated percentage range by weight of minor components.

Parkland GEO

GENERAL TERMS AND CONDITIONS

The use of this attached report is subject to acceptance of the following general terms and conditions.

1. STANDARD OF CARE - In the performance of professional services, ParklandGEO will use that degree of care and skill ordinarily exercised under similar circumstances by reputable members of its profession practicing in the same or similar localities. No other warranty expressed or implied is made or intended by this agreement or by furnishing oral or written reports of the findings made. ParklandGEO is to be liable only for damage directly caused by the negligence of ParklandGEO.

2. INTERPRETATION OF THE REPORT - The CLIENT recognizes that subsurface conditions will vary from those encountered at the location where borings, surveys, or explorations are made and that the data, interpretations and recommendation of ParklandGEO are based solely on the information available to him. Classification and identification of soils, rocks, geological units, contaminated materials and contaminant quantities will be based on commonly accepted practices in geotechnical consulting practice in this area. ParklandGEO will not be responsible for the interpretation by others of the information developed.

3. SITE INFORMATION - The CLIENT agrees to fully cooperate with ParklandGEO and provide all information with respect to the past, present and proposed conditions and use of the Site whether specifically requested or not. The CLIENT acknowledges that in order for ParklandGEO to properly advise and assist the CLIENT in respect of the investigation of the Site, ParklandGEO is relying upon full disclosure by the CLIENT of all matters pertinent to an investigation of the Site.

   Where specifically stated in the scope of work, ParklandGEO will perform a review of the historical information obtained or provided by the Client to assist in the investigation of the Site unless and except to the extent that such a review is limited or excluded from the scope of work.

4. RIGHT OF ENTRY - The CLIENT is responsible for ensuring that ParklandGEO is provided unencumbered access to the property to the extent necessary for ParklandGEO to complete the scope of work to ParklandGEO's satisfaction. The CLIENT is solely responsible for obtaining permission and permits for ParklandGEO to enter onto the subject site, including informing tenants. The CLIENT shall also provide ParklandGEO with the location of all underground utilities and structures on the subject site, unless otherwise agreed to in writing. While ParklandGEO will take all reasonable precautions to avoid and minimize any damage to any sub-terrain utilities or structures, the CLIENT agrees to hold ParklandGEO harmless for any damage to any sub-terrain utilities or structures or any damage occasioned in gaining access to the subject site.

5. COMPLETE REPORT - The Report is of a summary nature and is not intended to stand alone without reference to the instructions given to ParklandGEO by the CLIENT, communications between ParklandGEO and the CLIENT, and to any other reports, writings or documents prepared by ParklandGEO for the CLIENT relative to the specific Site, all of which constitute the Report. The word "Report" shall refer to any and all of the documents referred to herein. In order to properly understand the suggestions, recommendations and opinions expressed by ParklandGEO, reference must be made to the whole of the Report. ParklandGEO cannot be responsible for use of any part or portions of the report without reference to the whole report. The CLIENT agrees that any and all reports prepared by ParklandGEO shall contain the following statement:

   > "This report has been prepared for the exclusive use of the named CLIENT. Any use which a third party makes of this report, or any reliance on or decisions to be made based on it, are the responsibility of such third parties. PARKLAND GEO-ENVIRONMENTAL LTD. 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 CLIENT agrees that in the event that any such report is released to a third party, such disclaimer shall not be obliterated or altered in any manner. The CLIENT further agrees that all such reports shall be used solely for the purposes of the CLIENT and shall not be released or used by others without the prior written permission of ParklandGEO.

6. LIMITATIONS ON SCOPE OF INVESTIGATION AND WARRANTY DISCLAIMER

There is no warranty, expressed or implied, by ParklandGEO that:

a) the investigation shall uncover all potential contaminants or environmental liabilities on the Site; or

b) the Site will be entirely free of all contaminants as a result of any investigation or cleanup work undertaken on the Site, since it is not possible, even with exhaustive sampling, testing and analysis, to document all potential contaminants on the Site.

The CLIENT acknowledges that:

a) the investigation findings are based solely on the information generated as a result of the specific scope of the investigation authorized by the CLIENT;

b) unless specifically stated in the agreed Scope of Work, the investigation will not, nor is it intended to assess or detect potential contaminants or environmental liabilities on the Site;

c) any assessment regarding geological conditions on the Site is based on the interpretation of conditions determined at specific sampling locations and depths and that conditions may vary between sampling locations, hence there can be no assurance that undetected geological conditions, including soils or groundwater are not located on the Site;

d) any assessment is also dependent on and limited by the accuracy of the analytical data generated by the sample analyses;

e) any assessment is also limited by the scientific possibility of determining the presence of unsuitable geological conditions for which scientific analyses have been conducted; and

f) the analytical parameters selected are limited to those outlined in the CLIENT's authorized scope of investigation; and

g) there are risks associated with the discovery of hazardous materials in and upon the lands and premises which may inadvertently discovered as part of this investigation. The CLIENT acknowledges that it may have a responsibility in law to inform the owner of any affected property of the existence or suspected existence of hazardous materials. The CLIENT further acknowledges that any such discovery may result in the fair market value of the lands and premises and of any other lands and premises adjacent thereto to be adversely affected in a material respect.

7. CONTROL OF WORK SITE AND JOBSITE SAFETY - ParklandGEO is only responsible for the activities of its employees on the jobsite. The presence of ParklandGEO personnel on the Site shall not be construed in any way to relieve the CLIENT or any contractors on Site from their responsibilities for Site safety. The CLIENT undertakes to inform ParklandGEO of all hazardous conditions, or possible hazardous conditions which are known to him. The CLIENT also recognizes that the activities of ParklandGEO may uncover previously unknown hazardous materials and that such a discovery may result in the necessity to undertake emergency procedures to protect ParklandGEO employees as well as the public at large and the environment in general. The CLIENT also acknowledges that in some cases the discovery of hazardous conditions and materials will require that certain regulatory bodies be informed and the CLIENT agrees that notification to such bodies by ParklandGEO will not be a cause of action or dispute.