Appendix A

Available Information Documents

GEOTECHNICAL INVESTIGATION REPORT
RCMP BUILDING

LOT 93, BLOCK 1, PLAN 151 0788
COALDALE, ALBERTA

PREPARED FOR

ROYAL CANADIAN MOUNTED POLICE
OTTAWA, ONTARIO

Royal Canadian Mounted Police
Gendarmerie royale du Canada

PREPARED BY

PARKLAND GEOTECHNICAL CONSULTING LTD.
LETHBRIDGE, ALBERTA

ParklandGEO

PROJECT NO. LE0114

JULY 14, 2017

Geotechnical, Environmental and Materials Engineering

Red Deer · Sherwood Park · Grande Prairie · Calgary · Fort McMurray
Peace River · Medicine Hat · Lethbridge · Estevan

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1.0 INTRODUCTION

The Royal Canadian Mounted Police (RCMP) is proposing to construct a new headquarters building in Coaldale, Alberta. Parkland Geotechnical Consulting Ltd. (ParklandGEO) was requested to perform a geotechnical investigation for the proposed development. The scope of work was outlined in ParklandGEO's proposal dated May 19, 2017 (File # LP1248REV1). Authorization to proceed with this geotechnical investigation was given by Mr. Jordan McKenna, Senior Procurement Officer, in an email dated June 5, 2017 This report summarizes the results of the field and laboratory testing programs and presents geotechnical recommendations including foundation design recommendations for the building and radio tower, excavation and backfill, seismic classifcation, asphalt concrete pavements, concrete slabs on grade, cement type, shoring design parameters, modulus of subgrade reaction, site grading, drainage and frost protection.

2.0 PROJECT BACKGROUND

2.1 SITE DESCRIPTION

The proposed RCMP Building is located on a large L-shaped lot at Lot 93, Block 1, Plan 151 0788, on the east side of Coaldale, Alberta. The site location is shown on the Key Plan, Figure 1. Access to the site was from Highway 3 to the north and 8th Street to the west of the property. The subject property is relatively level with an elevation difference of less than 1.2 m between borehole locations. At the time of the field investigation, the site consisted of a bare lot north of a storm water pond with a sidewalk on the north side and a chain link fence separating the site from a utility pump station and storm pond. The site was stripped and levelled during the recent subdivision development. The only vegetation was weeds and sparse grass. The residential subdivision of Parkside Estates is located south of the subject property. A Site Plan illustrating the layout of the proposed development is shown on Figure 2. Ground elevations are given on the site plan. The subject property is bordered by similar commercial properties to the east, the 19A Avenue service road and Highway 3 to the north and residential subdivisions to the south and west of the property. The site and surrounding area are shown on the Aerial Plan, Figure 3.

2.2 DEVELOPMENT HISTORY OF THE AREA

During drilling, a thick surficial fill was encountered at the site. The history of the site was reviewed to determine thickness of the fill and when it was placed. Included in Figures 5 through 8 are a series of four aerial photographs showing the site from 1989 when the area was active agricultural land and relatively level.

Between 1989 and 1996, Range Road 201 (8th Street) which runs west of the subject property was realigned in order to intersect Highway 3 at a perpendicular angle as shown in Figure 6. This roadway appears to have been constructed using material excavated from the storm water retention pond. The subject property has been developed along with the adjoining lot to the east and a berm on the east side north of the pond. It appears from the aerial photo that the original lot elevation were about 1 m lower than the top of the berm, 8th Street and the service road to the north (19A Avenue) which runs parallel to Highway 3. The Town of Coaldale pump house is also in place by 1996 north of the storm pond and east of the southeast corner of the lot in a utility right-of-way.

Prior to 2003 Phase 1 of Parkside Estates was being graded and the storm pond was being enlarged. A storm sewer was being built from the northwest corner of the site to the northwest corner of the storm pond. This right of way is remains on the site to the present and was located by the Town of Coaldale prior to drilling. Material from the pond and storm line excavation was placed on the north portion of the lot in the 2003 photo. Anecdotal info from the drilling contractor suggests the excavation from the initial storm pond was used to raise the lots up the berm and rough grading of Parkside Estates. The south west portion of the lot west of the pond remained more or less at the original grade with little to no fill placed. The excess material was hauled to a spoil pile in the north side of town.

By 2009, the site was developed to present grades and Parkside Estates was fully developed with houses. The changes that have taken place since 2009 include the paving of 8th Street and 19A Avenue, the storm pond has been enclosed in chain link fence and landscaped. Information from the landscaping firm indicates that the landscaping around the pond was done in 2015. From the information obtained from the aerial photos and interviewing local sources, and our survey, the maximum amount of fill on the site is about 1 m and it was placed between 2003 and 2009.

2.3 PREVIOUS GEOTECHNICAL INVESTIGATIONS

A previous geotechnical investigation was performed at the site by Tetra Tech EBA circa 2014. The engineer performing the investigation was Mitchell Van Orman who now is with ParklandGEO. During that investigation, the general soil profile encountered was high plastic lacustrine clay overlying medium plastic clay till. There are four, intact, 25 mm PVC standpipes on the site from that investigation. Water levels taken during our geotechnical investigation show similar groundwater elevations to the existing standpipes and these standpipes were part of our site survey to allow all of the standpipes to be monitored and are labelled "TT" in Table 1.

2.4 PROJECT DESCRIPTION

The proposed project will involve the construction of a 2,500 m^2, single storey masonry building with paved parking lots on the west and east sides. There will also be a radio communication tower and several outbuildings (garages). Only general information was known about the building design and location of the radio tower at the time of this investigation. Foundation loads for the proposed building are expected to be light to moderate. No basement has been proposed. Site traffic on the asphalt areas will be light passenger vehicle and the occasional loading or waste management truck. Photographs taken at the time of the field investigation are presented in Figure 4.

3.0 FIELD AND LABORATORY PROGRAMS

On June 19 and 20, 2017, eight deep boreholes and five shallow boreholes were drilled at the site to depths ranging from 8.0 m to 13.0 m below grade. The locations and ground surface elevations of the boreholes are shown on the Site Plan, Figure 2. The following sampling and testing procedures were followed during the field program:

1. Prior to mobilizing the drilling rig, ParklandGEO personnel contacted Alberta One Call to verify the drill site was clear of underground utilities. The proposed locations of the 13 boreholes were located by ParklandGEO personnel on June 6, 2017.

2. The boreholes were drilled using a truck-mounted power auger drilling rig with 150 mm diameter solid stem augers. The drill rig was operated by Chilako Drilling Ltd.

3. Drilling operations were monitored by ParklandGEO personnel. The soil encountered was visually examined during drilling and logged according to the Modified Unified Soil Classification System.

4. Soil samples were collected from auger cuttings at 1.0 m intervals in order to determine the soil/moisture profile and from other selected depths for other testing. Soil samples were also obtained from Standard Penetrations Tests (SPTs), which were performed at selected depth intervals.

5. At the completion of drilling, 25 mm hand-slotted PVC standpipes were installed in select boreholes. All boreholes were backfilled with auger cuttings and bentonite.

6. All soil samples were returned to ParklandGEO's Lethbridge laboratory for further testing. The results of the laboratory testing are shown on the borehole logs in Appendix A and the individual test results are presented in Appendix B. The laboratory testing program consisted of moisture contents, Atterberg Limits, hydrometers, and sulphate testing.

7. Groundwater conditions were noted during drilling. Groundwater level measurements were taken on June 27, 2017, seven days after drilling.

8. The locations and elevations of the boreholes were surveyed by ParklandGEO using a Trimble GeoXH 2008 Series GPS receiver and a Trimble Tornado GPS antenna. The estimated post data correction vertical accuracy of this equipment is ± 10 cm. The ground surface elevations at the borehole locations are referenced to a geodetic datum. A manhole cover on 19A Avenue and a fire hydrant base flange were surveyed to be used as local benchmarks.

9. On June 30, 2017, three Shelby Tube samples were taken at the site at a depth of 0.3 m for dry unit weight determination on the suspected fill material. The location of the holes samples were next to Boreholes 7, 12 and in the proposed borehole location for Borehole 14 which is about halfway in between Boreholes 9 and 2. Three in situ dry densities

were performed at the site in the building footprint area using a Troxler 3430 Moisture Density Gauge.

4.0 SUBSURFACE CONDITIONS

The soil profile at the site was, in descending order: clay fill, clay, clay and clay till. The fill was consistently about 1 m thick in all boreholes except Boreholes 7 and 8 at the south west area of the site, just west of the storm pond. This area was considered to be close to original grade (ie. Minimal fill). The fill was placed between 2003 and 2009 during the rough grading of Parkside Estates Phase 1 and expansion of the storm water pond. The till was made up of material from the pond and there was no buried topsoil layer so the transition between the clay fill and lacustrine clay was poorly defined. The clay till deposits were very consistent in depth and extended beyond the depth of drilling six of the eight deep boreholes. At Boreholes 5 and 12 sand deposits were encountered below the till. This soil profile is considered typical for the Coaldale area. Detailed soil conditions encountered at each of the borehole locations are described in 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 TOPSOIL

There was little to no topsoil encountered at the site as the area appeared to be filled and rough graded using local native materials likely from excavation of the nearby storm water retention pond. There were only sparse grass and weeds growing on the site with no defined topsoil layer. No buried organics were encountered in any of the boreholes but it is possible that they may be found at other locations on the site due to the fill that was placed. As there was no evidence of buried organics, it suggests that the site was properly stripped prior to the placement of the fill in between 1996 and 2009. The fill that was placed appeared to be void of organic materials.

4.2 CLAY FILL

A layer of clay fill or disturbed surficial clay was encountered in all of the boreholes. Our most reliable information is the aerial photographs and anecdotal information from local contractors. The depth of this fill was not distinct and appeared to vary between boreholes with a maximum estimated depth of 1.1 m. The fill may be deeper between borehole locations. At the time of investigation, no compaction testing records were available for the site.

The moisture content of the material was typically dry to moist ranging from 8 percent to 15 percent. The average moisture content was about 12 percent which would be below the Optimum Moisture Content (OMC) for the material. A sample taken at the interface between the fill and the clay had a PI of 23 indicating that it was a medium plastic clay similar to the clay till layer that would have been excavated from the storm pond. The material had a stiff to very stiff consistency.

The fill appeared to be uniform and there was no observation of buried or mixed organics at the site suggesting that the material was placed in a controlled manner. Soil densities appear to be between 1730 and 1800 kg/m^3 which are the expected ranges for lacustrine clay and clay till materials, respectively. The average moisture content was slightly below the estimated OMC for the material. The material has been in place for over 8 to 10 years, so it is considered to be fully consolidated under self-weight.

4.3 CLAY

The soil below the surficial clay fill was lacustrine clay. Although it was difficult to distinguish, the depth to the native clay was believed to be 0.4 to 1.1 m at the borehole locations. The clay was present to depths of 2.7 to 3.5 m. The material was medium plastic with a Plasticity Index (PI) of 23 percent indicating a medium plastic clay. The Standard Penetration Test (SPT) "N" values of the material ranged from 12 to 29 with an average of 21, indicating a very stiff consistency. The moisture content ranged between 9 and 26 percent with an average of about 17 percent which is considered to be at or slightly about the estimated OMC for the material.

4.4 CLAY TILL

Glacial clay (till) deposits were encountered in all of the boreholes and extended to a depth greater than the maximum 13 m depth drilled in all but two of the eight deep boreholes. The till deposits were a variable mixture of clay, silt and sand, with traces of gravel, occasional rust stains, and coal inclusions. Thin sand lenses and were noted within the till with only minor sloughing occurring during drilling in a few of the holes. Although not encountered, cobbles and boulders are common in the local till.

The upper till from 3 m to 8 m below grade was high plastic with a plasticity index of 44 percent and a Liquid Limit (LL) of 66 percent. The SPT "N" values ranged from 8 to 24 blows per 150 mm of penetration with an average of 14, indicating a stiff consistency. The moisture contents ranged from about 15 to 31 percent which is considered to be at or above the OMC for these deposits. Due to the clay being high plastic, it may have swelling issues and steps to mitigate changes in moisture content will need to be implemented to minimize these issues. See Section 6.4 for a discussion of high plastic clay issues.

The lower till from 8 m to 13 m below grade was low to medium plastic with a plasticity index of 18 percent. The SPT "N" values ranged from 15 to 33 blows per 150 mm of penetration with an average of 23, indicating a very stiff consistency. The moisture contents ranged from about 14 to 23 percent which is considered to be at or above the OMC for these deposits.

4.5 SAND

Sand was encountered in Borehole 5 and 12 below 12.5 m. The sand was silty, clayey, compact and poorly graded. The moisture content was 17 percent or saturated and it had an N values ranging from 8 to 18. A grain size analysis performed on a sample from Borehole 5 indicated that the material is a silty sand. The sand was saturated with a moisture content of 17 percent which is considered to be above the estimated OMC for the material.
This sand layer is about 5 m thick and can be found below the clay till for a large area of Coaldale and may be pre-glacial according to our drilling contractor.

5.0 GROUNDWATER CONDITIONS

Groundwater seepage and sloughing was observed at the borehole locations during and after drilling. Groundwater level measurements were taken on June 27, 2017, seven days after drilling using the 10 standpipes installed by ParklandGEO and the three existing standpipes from the previous site investigation. The existing wells have been in place for several years and should be stabilized. The following table summarizes the observed groundwater conditions.

TABLE 1
GROUNDWATER MEASUREMENTS

Borehole	Ground Elevation (m)	Upon Completion		June 27, 2017	
		Borehole Depth (m)	Groundwater Level (mbg)	Groundwater Level (mbg)	Groundwater Elevation (m)
1	861.36	8.0	Dry	7.93	853.43
2	861.11	8.0	Dry	7.36	853.75
3	860.91	12.5	9.5	3.88	857.03
4	861.08	12.0	11.3	No well	-
5	861.02	12.5	12.0	3.84	857.18
6	861.28	12.5	Dry	3.92	857.36
7	860.34	8.0	Dry	7.85	852.49
8	861.01	8.0	Dry	No well	-
9	861.48	12.5	10.7	4.65	856.83
10	861.06	13.0	12.9	3.54	857.52
11	861.36	12.5	Dry	No well	-
12	861.40	12.5	12.0	3.68	857.72
13	861.00	8.0	Dry	5.30	855.70
TT2	861.34	9.6	-	4.02	857.32
TT5	860.73	3.5 *	-	4.04	856.69
TT6	860.80	7.7	-	3.50	857.30

*The standpipe in TT5 was damaged or plugged at 3.5 m below grade.

The observed groundwater conditions are considered to be near the seasonal normal. Sand lenses which may be encountered within the till are recharged by infiltration of precipitation (rain and snow melt). Groundwater elevations are expected to fluctuate higher on a seasonal basis and will be highest after periods of heavy or prolonged precipitation and snow-melt. Due to the low permeability of the clay subgrade the response to heavy precipitation is expected to be slow. The subgrade will be susceptible to perched groundwater conditions on a seasonal basis during the spring and summer months. Perched conditions will dissipate over time as the groundwater evaporates or infiltrates down to the groundwater table. Groundwater seepage is not expected for shallow excavations at this site. These sand lenses are not usually interconnected, so the volumes of groundwater encountered will be dependent on seasonal conditions and the permeability of the soil layers intercepted by excavations.

There is large storm water retention pond south of the subject property which due to its proximity and lower elevation should have the effect of keeping groundwater levels relatively low indefinitely due to its draw down of the water table. The high water mark of the pond is 856.63 m and the low water level is 855.19 m which is between 5 and 6 m lower than the existing site elevation. The water level in the pond at the time of the investigation was 855.19 m but will fluctuate with precipitation runoff.

6.0 DISCUSSION AND RECOMMENDATIONS

6.1 GEOTECHNICAL EVALUATION

The proposed development will involve constructing a 2,500 m^2 single storey building and radio tower on the site along with paved parking areas on the west and east sides of the building and some miscellaneous out buildings. Foundation loads for the proposed buildings are expected to be light to moderate and no basements have been proposed. The area of the parking lot that provides access to the garbage bin will have heavy truck traffic and the remaining areas of the asphalt areas will only have passenger vehicle traffic. There is a storm sewer line which crosses from the northwest corner of the site to an outfall on the northwest corner of the storm pond that was installed circa 2003. This fill is likely self-consolidated under its own weight but may be thicker than other areas on the site. This right of way is not in the building footprint. The main geotechnical considerations for the project include:

1. The entire site is covered with suspected fill material which is a silty clay and clay till mixture that may be material excavated from the storm water retention pond. The material is very stiff and appears to be engineered fill placed when the lot was developed. At the time of the investigation, no compaction testing records were available for the site. The fill appears to have been placed in a controlled manner. The fill was considered to be uniform and has been in place over eight years, so it is expected to be fully consolidated under self-weight. With proper preparation the fill is considered to be suitable for support of floor slabs, but additional assessment would be required to consider the use of fill for support of shallow foundations.

2. The static groundwater level at this site was estimated to be about 3.5 to 5 m below existing grade. Only minor sloughing and free water was observed relatively deep in the boreholes. Groundwater elevations are expected to fluctuate higher after periods of heavy precipitation or snow-melt. Based on these observations, deep trench excavations at the site above seepage zones may require relatively flat side slopes and dewatering measures to handle groundwater seepage. Conventional pumping arrangements from collector sumps should be suitable for typical excavations at the site.

3. The upper clay till is high plastic and may have swelling and/or shrinkage issues with changes in moisture content. The till is over 3 m below grade and is covered by clay fill and native clay. If the till is exposed, care needs to be exercised during construction to minimize drying or wetting of these deposits. Since till was used as fill material the potential for pockets of near surface swelling clay is possible. Mixing of this till material with lower plastic clay deposits will help mitigate swelling potential.

4. Due to the fine grained nature of the surficial soils, subgrade conditions may be adversely impacted by wet weather and seasonal high groundwater levels including perched groundwater conditions. Shallow groundwater in fine grained silty soils are a concern because of the potential for groundwater to "pump up" to surface due to repetitive construction traffic resulting in a significant weakening/failure of the subgrade. Site preparation measures will be significantly impacted by wet weather.

5. The level of subgrade support from the upper silty clay soils will be relatively low. 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.

6. Moderate and light duty asphalt concrete pavement designs will be required. The area of the parking lot used to access the garbage bin will have moderate truck traffic. All other areas will be light duty and only have light passenger vehicle traffic.

7. The foundation conditions are considered suitable for conventional footings or deep pile foundations. If footings are considered the recommendation would be to found all footings on the native soil below the fill. This may result in relatively deep footings across the building. The clay profile would be suitable for several pile options including cast-in-place (CIP) concrete, driven steel and steel screw piles. CIP concrete piles are considered to be a common pile option in this area. Foundation recommendations are given for footings and CIP concrete piles. Recommendations for other pile options can be given upon request.

8. Geotechnical parameters are required for the design of a radio communication tower. At the time of the investigation, limited information was available about the height or style of the tower. However, it is expected that the tower may be subject to axial compression, uplift and lateral loads.

6.2 SITE PREPARATION

Recommendations for site preparation within the building footprint areas will be dependent on the decision to construct grade supported or structurally supported floor slabs. Due to the depth of the existing clay fill soils in the building footprint, it is assumed that full replacement of the existing fill to support slabs is not an economically viable option. Therefore, the two most practical options are:

6.2.1 Structurally Supported Floor Slab

The structural floor slab alternative is the least time dependent alternative available and performance levels for the slab can be engineered. The use of a structurally supported slab will also allow a lower level of compaction to be used on the exposed subgrade. A structural floor slab may be considered across the entire building footprint. If a structural floor slab is proposed, fill required to bring the site up to grade must be capable of supporting short-term loads for concrete forms. The fill required to bring the site up to grade should be well graded select sand, gravel or low to medium plastic clay. Fill material should be placed to a uniform density of 95 percent of Standard Proctor Maximum Dry Density (SPMDD).

6.2.2 Partial Removal and Replacement of the Existing Fill

Please not, these two options are not equivalent. The option with the most predictable performance and the fewest risks for this site is a structurally supported floor slab. If the Owner is willing to accept higher risk associated with vertical slab movement and non-uniform support, the partial replacement option may be considered. This option requires heavy compaction effort to try and identify non-uniformities and improve the subgrade during construction.

The existing fill in the building footprint may be 1.0 to 1.5 m thick. However, the fill appears to be fully consolidated and uniform with sufficient strength and density to support floor slabs provided some preparation measures are undertaken to verify the fill condition and improve the subgrade. If the Owner is willing to accept some risk of vertical slab movement, it is considered reasonable to undertake a deep, heavy preparation of the existing subgrade. It is also recommended to increase the base layer thickness below the slab.

6.2.3 Building Areas- Partial Replacement Option

It is recommended to remove the upper 450 mm of the existing clay fill and replace it with a select engineered fill compacted to at least 99 percent of SPMDD. It may be possible to re-use the excavated fill provided it can be moisture conditioned to 0-2 percent above OMC and completed to the specified density in 150 mm lifts. If this is not possible, the use of select granular fill will be required. Granular fill should be compacted in thin lifts at a moisture content 0-2 percent below OMC. The building pad should extent to a distance greater than the depth of new fill.

It should be understood that the subgrade may be sensitive to disturbance and subgrade conditions may be adversely impacted by wet weather and groundwater levels. Therefore, the fill removal and replacement option may experience compaction problems which will impact future slab performance.

6.2.4 Pavement Areas

The proposed development includes construction of parking areas. In general, the discussion for buildings, as given above, applies to parking areas and the best solution is to replace the old fill with engineered fill. Since the performance expectations and vertical tolerances are not usually as stringent for pavement areas, it is probably practical to re-establish the parking areas surface on the existing fill, provided subject to normal site preparation activity.

The level of subgrade support for the existing subgrade will be relatively low and may require additional localized improvement to support pavement structures. The affected pavement areas should be inspected prior to filling to identify any soft areas. Soft areas should be sub-cut and replaced with a suitable fill material to a depth sufficient to support construction traffic.

6.2.5 Materials

Fill required to bring the site up to grade should be well graded select sand or gravel. The existing clay fill materials are considered to be marginally suitable for use as engineered fill, and will require significant moisture conditioning (ie drying) 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
ALBERTA TRANSPORTATION 80 MM GRAVEL

Sieve Size (mm)	Percent Passing by Weight
80	100
50	55 – 100
25	38 – 100
16	32 - 85
5	20 – 65
0.315	6 - 30
0.08	2 - 10

6.2.6 Site Grades

Surface water should be drained away from the building site as quickly as possible, both during and after construction. Site drainage should be directed away from the foundation walls. It is recommended to provide a 5 percent back slope from the building for a distance of at least 3 m. Roof and other drains should discharge well clear of the building. The slope of exterior backfill should be checked periodically to verify water is shed away from the building. If the backfill settles causing water to pond against the foundation wall, the surface should be re-graded.

6.3 ALBERTA BUILDING CODE

In accordance with the most recent version of the 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).

6.3.1 Ultimate Limit States (ULS)

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. The following GRF values should be used for foundation design:

TABLE 3
LSD GEOTECHNICAL RESISTANCE FACTORS

GEOTECHNICAL CASE	RESISTANCE FACTORS
DEEP FOUNDATIONS (PILES)	
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
Uplift resistance by semi-empirical analysis and in-situ test data	0.3
Uplift resistance from analysis of static load test results	0.4
SHALLOW FOUNDATIONS (FOOTINGS)	
Vertical resistance by semi-empirical analysis and in-situ test data	0.5
Sliding based on friction	0.8

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

6.3.2 Serviceability Limit States (SLS)

The SLS case 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 within tolerable limits under service loads (ie. settlement, lateral deflection, etc.). Typically, the foundation loads, configurations, and serviceability tolerances have to be known to properly determine geotechnical SLS resistance values. In some foundation cases, such as small footings, basic assumptions can be used to provide preliminary SLS resistance values under specific stated conditions.

For pile foundations under axial loading conditions the SLS resistance is addressed by determining the limiting load to keep foundation settlements within tolerable limits. Tolerable total and differential settlements should be verified by the structural engineer, but for normal buildings the tolerable limit of total settlement for foundations is typically about 25 mm. For piles, less than 25 mm of settlement is usually required to mobilize the ultimate resistance. Therefore, the SLS are not expected to govern the foundation design unless very strict settlement tolerances are required (i.e. less than 10 to 15 mm of settlement). The settlement potential of the proposed piles may be checked once pile design and loading conditions are finalized.

6.3.3 Seismic Classification

The National Building Code of Canada requires buildings to be designed to resist a minimum earthquake force. The formula for obtaining minimum earthquake force is dependent on several factors including Foundation Factors (F_a and F_v) which should be determined using a Site Class of D for this site (Table 4.1.8.4.A). The subgrade soil is a stiff silt overlying a stiff to hard clay till with an average undrained shear strength of about 350 kPa above 8 m below grade and 500 kPa below 8 m below grade.

6.4 FOOTINGS

Footings are considered to be suitable for foundation loads bearing on the native clay soils at this site. Based on current information the existing fill is considered unsuitable for support of footings without further review. Footings founded on the native clay deposits with 2.5 of grade may be designed based on the Ultimate Limit States (ULS) and the Serviceability Limit States (SLS) using the bearing resistance values in the following table:

TABLE 4
BEARING RESISTANCE FOR FOOTINGS*

Type	ULS (kPa)		SLS (kPa)
	Ultimate	Factored	
Strip Footing	350	175	140
Spread Footing	425	200	170

* For footings bearing on native site soils within 2.5 m of existing grade.

The "factored" ULS resistance given above has been calculated by multiplying the unfactored bearing capacity values by a geotechnical resistance factor of 0.5, in accordance with the building code as summarized in Section 6.3. The SLS bearing resistance values given above are based on limiting the settlement to 25 mm or less, and are applicable to footings with a maximum dimension of 1.5 m wide or 2.0 x 2.0 m^2. If very strict settlement tolerances are required or if larger footings are proposed, the footing sizes and settlement potential should be reviewed. If fill or soft foundation conditions are encountered at design depth, the use of a gravel mat below the footings should be considered to spread foundation loads to the subgrade.

The following recommendations should be adhered to for footing design and construction:

1. Footings should bear on undisturbed native inorganic soil or engineered gravel fill free of loosened material. Excavation of the footing trenches should be undertaken in a manner to minimize disturbance to the bearing surface.

   For protection against frost action, perimeter footings in continuously heated structures should be provided with a minimum depth of ground cover of 1.5 m. Interior footings should be founded at least 0.5 m below slab grade. Isolated footings and exterior footings in unheated structures will require at least 2.5 m of ground cover. Styrofoam insulation may be used to prevent frost penetration where adequate depths of ground cover cannot be economically provided. A standard frost protection figure is provided in Appendix A.

2. If footings are placed on an engineered gravel mat:

   - the bearing areas should be over-excavated a minimum of 200 mm below and to all sides of footings. Dependent on foundation level and localized fill or soft areas, it may be necessary to increase the thickness of the gravel mat. The gravel mat must extend beyond all sides of footings a distance equal to the gravel thickness to a maximum of 300 mm.

   - gravel should consist of select, well graded coarse gravel with a maximum particle size of 50 mm and less than 10 percent fines passing the 0.080 mm sieve.

   - the gravel should be compacted to at least 100 percent of SPMDD below the base of the footing. Over-compaction of the footing should be avoided.

   In the case of very soft or wet base conditions, the use of a filter fabric may be necessary to act as a separation barrier between the subgrade and the gravel backfill. The geotextile should be placed over the full base and sides of the excavation prior to backfilling.

3. The footing trenches should be protected against surface water run-off and seepage water through the use of conventional sumps and ditches, if required.

4. Foundation soils must not be allowed to freeze at any time during or after construction. Footings founded on frozen soils will settle when the founding soils are weakened by thawing.

5. Preparation of the bearing surfaces should be monitored by a qualified geotechnical engineer prior to placement of footings to verify that design criteria are met.

6.5 CAST-IN-PLACE (CIP) CONCRETE PILES – ULS DESIGN

Foundation are required for the main building, the out buildings, and the communication tower. Bored CIP concrete piles are considered to be well suited for this development. Bored CIP piles have successfully been used in this area for buildings and will be feasible, provided casing is made available and is used as required to control sloughing and/or seepage.

6.5.1 Radio Tower Construction Considerations

At the time of this investigation, information about the height, style and location of the proposed radio communication tower was not known. The client representative has indicated that a typical tower would have three legs and be about 25 m tall.

6.5.1 Axial Compression

Bored straight shaft or belled CIP concrete piles for this structure may be designed on the ULS skin friction or end bearing values given in the following table.

TABLE 5
CAST-IN-PLACE PILES - ULTIMATE RESISTANCE

Soil Type	Depth (m)	Ultimate Resistance Skin Friction (kPa)	Ultimate Resistance End Bearing (kPa)
Frost Zone	0 – 1.5	-	-
Fill	varies	0	-
Clay /Upper Till	1.5 – 4.5	50	-
Upper Till	4.5 – 8:0	40	700
Lower Till	Below 8.0	60	1100

The ultimate resistance values in this table are based on semi-empirical data, therefore the “factored” ULS resistance should be calculated by multiplying the ultimate values above by a geotechnical resistance factor of 0.4. The GRF for resistance to axial compression may be increased if the pile capacities are verified by a dynamic monitoring method.

Additional construction recommendations for CIP piles at the proposed site are provided below.

1. To resist uplift forces created by frost action, the minimum depth of straight shaft piles for heated structures should be 6.0 m below final grade and the minimum depth of straight shaft piles for unheated structures should be 7.0 m below final grade. If this embedment requirement cannot be met, the piles should be belled and insulated to provide the necessary protection against frost uplift. The minimum depth of belled piles is 4.5 m. This minimum embedment length does not apply to interior to interior piles in heating buildings.

2. Steel casing should be available on site during construction and should be used to prevent sloughing and groundwater seepage into the drill-hole.

3. Pile excavations should be filled with concrete within 2 hours upon completion of the pile excavation.

4. If belled piles are used:

    - The bell diameter should not exceed the shaft diameter by more than a factor of 3.0.
    - The roof of the bell should be 1H:1V (45 degrees) or steeper.
    - Bells should not be placed within sand/gravel layers.
    - The minimum distance from the underside of any sand layer to the roof of the bell should be 1.5 m.

5. Steel reinforcement should extend the full length of the pile and extend into the pile bells for end bearing piles and at least 6.0 m for straight shaft friction piles. The minimum recommended pile diameter is 400 mm.

6. All pile installations should be inspected by a qualified geotechnical engineer or technician to verify that design criteria are met or exceeded.

6.5.2 Uplift Resistance of Piles

The ultimate uplift resistance of bored concrete friction piles due to structural loads such as wind loads may be based on an ultimate skin friction values in Table 5. Since the values in this table are based on semi-empirical data, the "factored" resistance should be calculated by multiplying the ultimate values by a geotechnical resistance factor of 0.3 in accordance to the NBCC. Pile foundations which are required to resist uplift forces should be checked for resistance to both pullout and structural ability of the pile section to carry tensile stresses. Uplift loads due to frost and wind loads are not additive since the two load mechanisms are vastly different.

6.5.3 Frost Action on Piles

Pile shafts within the zone of frost of the subgrade will be subject to adfreeze forces which can cause frost jacking. The minimum pile depth given in Section 6.1.1 are provided to counter these forces.

Frost heave forces will also act on the underside of pile caps and grade beams with upward heaving pressure in the order of 1000 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 is 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 foundation walls 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.

6.6 STEEL SCREW PILES – ULS DESIGN

6.6.1 Axial Compression

Steel screw piles installed in the clay till are considered a feasible foundation option for this site. The ultimate unfactored load carrying capacity of a multiple helix screw pile in vertical compression may be calculated based on a rational method using resistance from the cylindrical shear acting on the soil cylinder between the top and bottom helixes and the bearing capacity developed by the bottom helix using the formula below.

$$Q_U = (N_C\ C_U\ A_H) + (S\ \pi\ C_U\ D_H\ (H_3 - H_1) \qquad \text{(Cohesive Soil - } H/D \geq 4)$$

Where:

Q_U = ultimate unfactored axial pile resistance (kN)
C_U = undrained shear strength of soil at the depth of the helix plates (kPa)
N_C = bearing capacity factor for cohesive soils (for $H/D > 4$)
H_3 = depth to bottom helix (m)
H_1 = depth to top helix (m)
A_H = gross area of the bottom helix (m^2)
D_H = average diameter of helix plates (m)
S = Spacing Factor (S = 1 if the space between helixes is <3D)

These formulas may also be used for a single helix screw pile since the second terms of the formula for the cylindrical shear between helixes is cancelled out (ie. $H_3 - H_1 = 0$). For Ultimate Limit States (ULS) design the ultimate resistance values calculated by this method are based on semi-empirical data, therefore the "factored" ULS resistance should be calculated by multiplying the ultimate resistance values above by a geotechnical resistance factor of 0.4. The GRF may be increased to 0.6 if a static load test program is performed.. The design parameters for the formulas given above are provided in the Table 6 below:

6.6.2 ULS Design for Uplift Resistance

The resistance of piles subjected to uplift loads may be calculated based on a rational method using resistance from the cylindrical shear acting on the soil cylinder between the top and bottom helixes and the weight of soil or bearing capacity developed by the upper helix. The recommended formula for ultimate uplift resistance for a multiple helix screw pile in cohesive soils (clay) soils where H/D ≥ 4 is given below.

$$Q_{UU} = (N_C\ C_U + \gamma`\ H)(A_H - A_S) + (\pi\ D_a\ C_U\ (H_3 - H_1)) \qquad \text{(Cohesive Soil – H/D} \geq 4)$$

Where: Q_{UU} = ultimate uplift pile resistance (kN)

$\gamma`$ = effective unit weight of soil (kN/m^3)

N_U = uplift bearing capacity factor for cohesive soils (for H/D > 4 use Nu = 9)

H = embedment depth of pile below final grade (m)

A_H = area of the top helix (m^2)

A_S = cross-sectional area of the shaft (m^2)

The recommended design parameters for screw piles are given in Table 6 in Section 6.5.2. Note, these formulas are dependent on meeting the stated H/D conditions for embedment since the uplift capacity increases with depth and the confining overburden thickness. If the given H/D requirements are not met, the method of determining the ultimate uplift capacity must be reviewed. The factored resistance for ULS analysis for axial tension (uplift) should be calculated by multiplying the unfactored values above by a geotechnical resistance factor of 0.3 in accordance with the NBCC.

TABLE 6
DESIGN PARAMETERS FOR SCREW PILES

	Silty Clay	Clay Till
Elevation (m)	858 - 861	> 858
Effective Unit Weight of Soil (γ' in kN/m^3)	18.0	19.0
Undrained Shear Strength, (C_u in kPa)	100	70-115
Internal Angle of Friction (ϕ)	25°	28°
Bearing Capacity Factor for Cohesive Soil (N_C) for H/D > 4	9	9

Screw piles should have a minimum embedment depth from ground surface to achieve sufficient frost jacking and tension loading resistance. Helices must not be founded within fresh fill materials. As a general recommendation for this site, the minimum screw pile depth of penetration should be considered as the maximum from the following three conditions: a) 4.5 m below ground surface; b) five times the helix diameter, or c) the design frost penetration depth plus one times the top helix diameter.

6.6.3 Other Design and Construction Recommendations

1. The maximum torque allowed during installation or "torque rating" of the proposed pile section must be considered during screw pile selection. The torque rating is an allowable value based on the ultimate torque strength or point of torsional fracture for the cross-sectional area of the steel in the pile shaft. The torque rating should be provided by the pile manufacturer. The estimated torque required to install the screw pile to the design resistance should not exceed the torque rating for the pile. For preliminary purposes, the minimum required torque may be taken as:

   $T = Q / K_T$ (Hoyt & Clemence, 1989)[1]

   Where: T = Minimum Torque Requirement (kN-m)
   Q = Design resistance of the pile (kN)
   K_t = empirical torque correlation factor = 10 m^{-1} for $D_S \geq 0.22$ m

2. The torque rating for the pile shaft should not be exceeded during installation of the pile. Using excessive torque on the pile may cause damage to the pile shaft which would require the pile to be replaced.

3. The minimum allowable centre to centre pile spacing should be taken as at least three helix diameters or 3.0 m. 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. The maximum vertical spacing for the helixes is 3 times the helix diameter for piles loaded in compression or tension. The method of determining pile capacity must be reviewed if this criterion is not met. The practical limit for helixes per pile is 4 for cohesive soil. Screw piles should have a minimum embedment depth from ground surface of 3.0 m to the top of the helix or the frost depth plus one helix diameter. The helix must be installed through any fills if present, and found at least 1.0 m into native undisturbed soils.

5. For frost uplift resistance, 1 helix diameter plus F distance is required to the upper helix where F equals 1.5 m for heated buildings and 2.1 m for unheated buildings.

6. Corrosion of the pile shaft in a partially saturated medium must be considered in selecting pipe shaft wall thickness. It is suggested to fill the pile shafts with concrete after installation to add strength to the pile shaft and reduce the corrosion potential inside the shaft of the pipe pile.

7. The soil strength is considered to be sufficient to provide lateral confinement to the pile shaft. Therefore, buckling of the shaft is not expected to be a concern.

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[1] Uplift Capacity of Helical Anchors in Soil", by H.R. Hoyt and S.P. Clemence, 1989, In Proceedings of the 12th International Conference on Soil Mechanics and Foundation Engineering. Rio de Janerio, Brazil, Vol 2, pp. 1019-1022.

8. 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 installation criteria are satisfied. The installation records should include a summary of the torque required to install the pile, particularly the average torque achieved during the last 1.5 m of pile installation.

6.6.4 Downdrag on Piles

Fills may be required for final site grading. Piles driven through new fill should be assumed to have a negative skin friction (down-drag) of 12 kPa acting on the section of pile shaft within the fill. Down-drag will diminish over time and will be eliminated when the fill is fully settled. Down-drag is not used in ULS design of piles, unless the potential downdrag loads exceed the governing structural live loads. Down-drag is an ultimate service load which may increase the amount of pile settlement. It should be checked as part of SLS analysis for settlement sensitive structures where the pile capacity is not governed by the ULS case. Down-drag is also applied in the structural ULS check on the pile section. A load factor of 1.25 should be applied for the structural check.

6.7 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.

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 a 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 (FHWA-IP-84-11, 1984). 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 used in these software programs is an iterative procedure for determining 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 (1964). 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 soils around the piles are expected to be native silty clay or clay till deposits.

6.8 GRADE SUPPORTED FLOOR SLABS

Grade supported floor slabs, supported by the engineered fill for the partial replacement option prepared as described in Section 6.2 are expected to perform adequately at this site. The magnitude of the expected vertical slab movements is considered to be within acceptable design tolerance. For floor slab design, a modulus of subgrade reaction (K_s) of 35,000 kN/m^3 is applicable for slabs placed on at least 150 mm of gravel base on the native subgrade. The following recommendations are provided for grade supported floor slabs in buildings which will be continuously heated:

1. Grade supported concrete slabs should be underlain with at least 300 mm of well graded, free draining, granular base with a maximum aggregate size of 50 mm and less than 10 percent passing the 0.080 mm sieve, compacted uniformly to 98 percent SPMDD.

2. Slabs should be constructed independently of all walls, columns and grade beams and may be tied into the grade beam with dowels at doorways. Alternatively, the slab can be tied into the grade beam at all points provided that a construction joint or cut is placed parallel to the grade beam and at a distance of approximately 2.0 m.

3. Slabs should be provided with construction joints or sawcuts consistent with local practice and should be reinforced with steel bars or equivalent wire mesh and dimensioned in accordance with the structural engineer's requirements. The reinforcing bars can be carried through the saw-cut joints.

4. Non-load bearing partitions should be designed to accommodate nominal vertical movements (approximately 25 mm). Mechanical equipment placed on floor slabs should be designed to permit some relevelling should the equipment be susceptible to small changes in level.

5. Piping and electrical conduit connections should be laid out to permit some flexibility, as vertical movement of such equipment as water meters, furnaces and electrical equipment may cause distress in the pipes. This provision is particularly important where there are short pipe runs between mechanical equipment and points where piping passes through the walls. Forced air heating ducts beneath the floor are not recommended, but if they are necessary, the ducts should be insulated with at least 50 mm of rigid insulation to prevent drying of subgrade soils.

6.9 EXCAVATIONS AND BACKFILL

Excavations will be required for foundations and underground utility installations. The latest edition of the Occupational Health and Safety Regulations of Alberta should be followed. Excavation side slopes are not expected to be able to stand near vertical for extended periods of time. Excavation side slopes should be cut back to 1H:1V from the bottom of the excavation. If space does not permit the slopes to be cut back, some form of temporary shoring must be installed.

For excavations through old fill, organic soil or groundwater, flatter side-slopes may be required. 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.

Recommendations regarding fill materials and compaction specifications given above in Section 6.2 should be followed for backfill. Compliance with compaction recommendations for exterior backfill around buildings is especially important, because poorly compacted backfill adjacent to foundation walls or grade beams will settle and may lead to ponding of surface water against foundation walls.

6.10 CONCRETE

The water-soluble sulphate concentration from the samples tested indicates severe potential for chemical attack of subsurface concrete. Therefore, Sulphate Resistant (Type HS) hydraulic cement is required for use in all subsurface concrete in contact with native soil at the site in accordance with CSA Standard CAN/CSA-A23.1-14. The recommended minimum 28-day compressive strength is 35 MPa with a water cement ratio of 0.4. All concrete exposed to a freezing environment either during or after construction should be air entrained.

6.11 SIDEWALKS AND EXTERIOR FLATWORK

The subgrade soils at the site are wet and fine grained and therefore susceptible to ice lens formation. Frost heave of exterior flatwork in front of doorways is a common problem in Alberta especially in areas shaded by buildings. Unprotected sidewalks dowelled into foundations often tip up due to heave rotating around the dowel connection, blocking doors, and promoting drainage towards the foundation wall. Unprotected sidewalks that are not dowelled into foundations may heave adjacent to the wall blocking doors and crushing any exterior wall facing not given enough clearance above the sidewalk.

The magnitude of heave in typical silty soils can range from 50 to 75 mm. If possible, exterior sidewalks should be moved away from foundation walls and exterior flatwork or sidewalks in front of doorways should be designed to minimize the impact of frost penetration on foundation walls and doors. The use of rigid structural insulation, heat tracing or a crushable, non-degradable void form material (so the void does not fill with water) should be considered in front of doorways. At least 75 mm of rigid insulation (Styrofoam HI or equivalent) should be placed below flatwork to restrict frost penetration into the subgrade soils. The insulation should taper out from the building, providing a gradual transition to unprotected subgrade. The exterior flatwork should slope away from the building and the sidewalk/building interface should be sealed to prevent seepage of surface runoff into the foundation soils.

6.12 FLEXIBLE ASPHALT PAVEMENT

Proposed pavement design sections are based on the assumption that the pavement will be constructed on a stable, prepared subgrade with a soaked California Bearing Ratio (CBR) 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. 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. This subgrade improvement gravel is placed together with the subbase.

Two flexible pavement designs are proposed for this site, one for light traffic in the parking areas; and one for moderate traffic on access roads and any truck loading areas. The assumed loading for moderate truck traffic is 50 heavy trucks per day over a 20-year design life; or about 1 x 10^6 Equivalent 80 kN Single Axle Loads (ESAL). To optimize pavements, the access ways around the main parking areas should be well defined to keep waste disposal trucks off of light traffic parking areas. If it is anticipated that traffic will exceed these levels, the design sections provided below should be reviewed.

TABLE 7
FLEXIBLE PAVEMENT DESIGN

	Light		Moderate	
Asphalt Concrete	75 mm	75 mm	100 mm	100 mm
Crush Base Gravel (minimum)	250 mm	150 mm	300 mm	150 mm
Granular Subbase (minimum)	-	200 mm	-	250 mm
Geosynthetic	Suggested		Yes	
Subgrade preparation (150 mm)	Yes		Yes	

The thickness of subbase gravel given above is considered to be the minimum requirement assuming no subgrade improvement is required. Some localized subbase gravel thickening may be required.

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 8
ASPHALT CONCRETE

Parameter	Specification
Stability (kN minimum)	8.0
Flow (mm)	2 – 3.5
Air Voids (percent)	2.8 – 3.2
VMA (percent)	14.0 – 16.0
VFA (percent)	70 – 80

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 Lethbridge specifications.

TABLE 9
RECOMMENDED AGGREGATE SPECIFICATIONS

	City of Lethbridge
Asphalt Gravel	Mix III
Crushed Base Gravel	Granular Base Course
Subbase Gravel	Granular Sub-Base

A copy of the City of Lethbridge 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.

6.13 FILTER FABRIC

As a general rule, if the subgrade is too soft or sensitive to undertake a conventional subgrade preparation, then the use of filter fabric should be considered. Since areas of the site have sensitive subgrade soils, the use of geotextile is required to act as a separation barrier between the subgrade and the gravel base. The suggested geotextile specification is:

TABLE 10
MINIMUM WOVEN FILTER CLOTH SPECIFICATION

Test Parameter	Specifications
Minimum Grab Tensile Strength	1100 N
Maximum Elongation at Break	25 percent
Minimum Mullen Burst Strength	2500 kPa
Minimum Tear Strength	400 N
Maximum Equivalent Opening Size	600 microns

Woven fabrics typically have more favorable stress/strain characteristics (30% elongation at failure) than non-woven filter fabrics (100 % elongation at failure). Therefore, the woven fabric will mobilize more strength as the subgrade deflects under construction traffic loads. Proposed geosynthetic filter fabrics should be reviewed based on their proposed end use. A slightly less robust geotextile could be given consideration if initial field performance ratings dictate. If sand fill is used on top of the native subgrade, a filter fabric is not required because there is limited potential for upward migration of fines and no need for a separation barrier.

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 parking areas, 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.

6.14 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. Adequate levels of inspection include: testing of engineered fill, review of all completed bearing surfaces for footings and full time inspection during construction of deep foundations.

7.0 LIMITATIONS

Geological conditions are variable. At the time this report was prepared, information on the subsurface conditions was available only at the borehole locations. Therefore, it was necessary to make certain assumptions concerning conditions between the borehole locations. The recommendations presented in this report and any subsequent correspondence, are based on an evaluation of information derived from thirteen boreholes. The conditions described are believed to be reasonably representative of the site. If conditions are noted during construction which are believed to be at variance with the conditions described in this report, this office should be contacted immediately.

This report has been prepared for the exclusive use of the **Royal Canadian Mounted Police** and their approved agents, for the specified application of the proposed development of the office building and radio tower in Coaldale, 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 Limitation Appendix of this report.

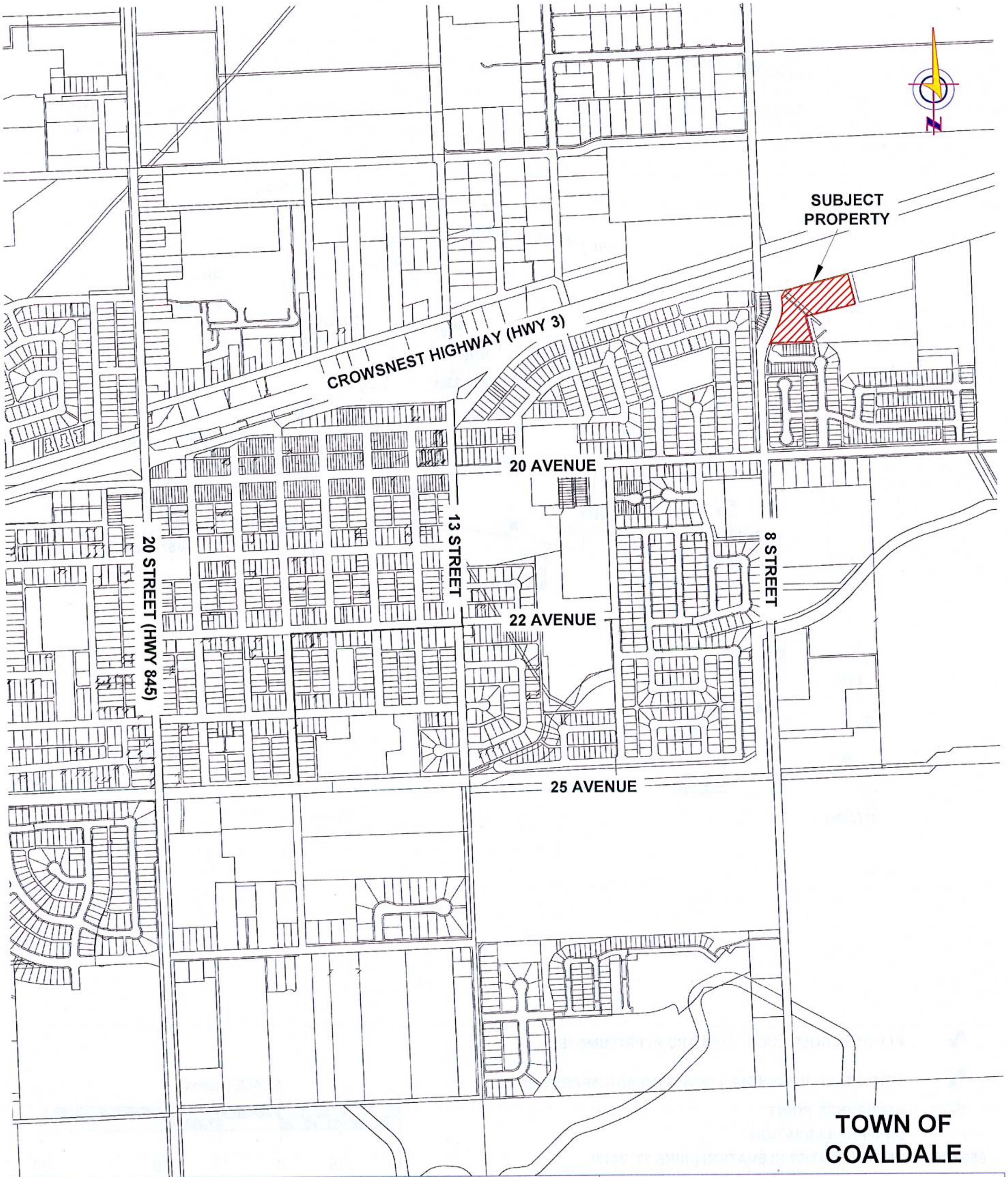
Respectfully submitted,
PARKLAND GEOTECHNICAL CONSULTING LTD.
APEGA Permit #07312

TREVOR H. BENSON
July 14, 2017

Trevor H. Benson, P.Eng.
Lethbridge Geo-Materials Manager

Reviewed by:
Mark Brotherton, P.Eng.

FIGURES



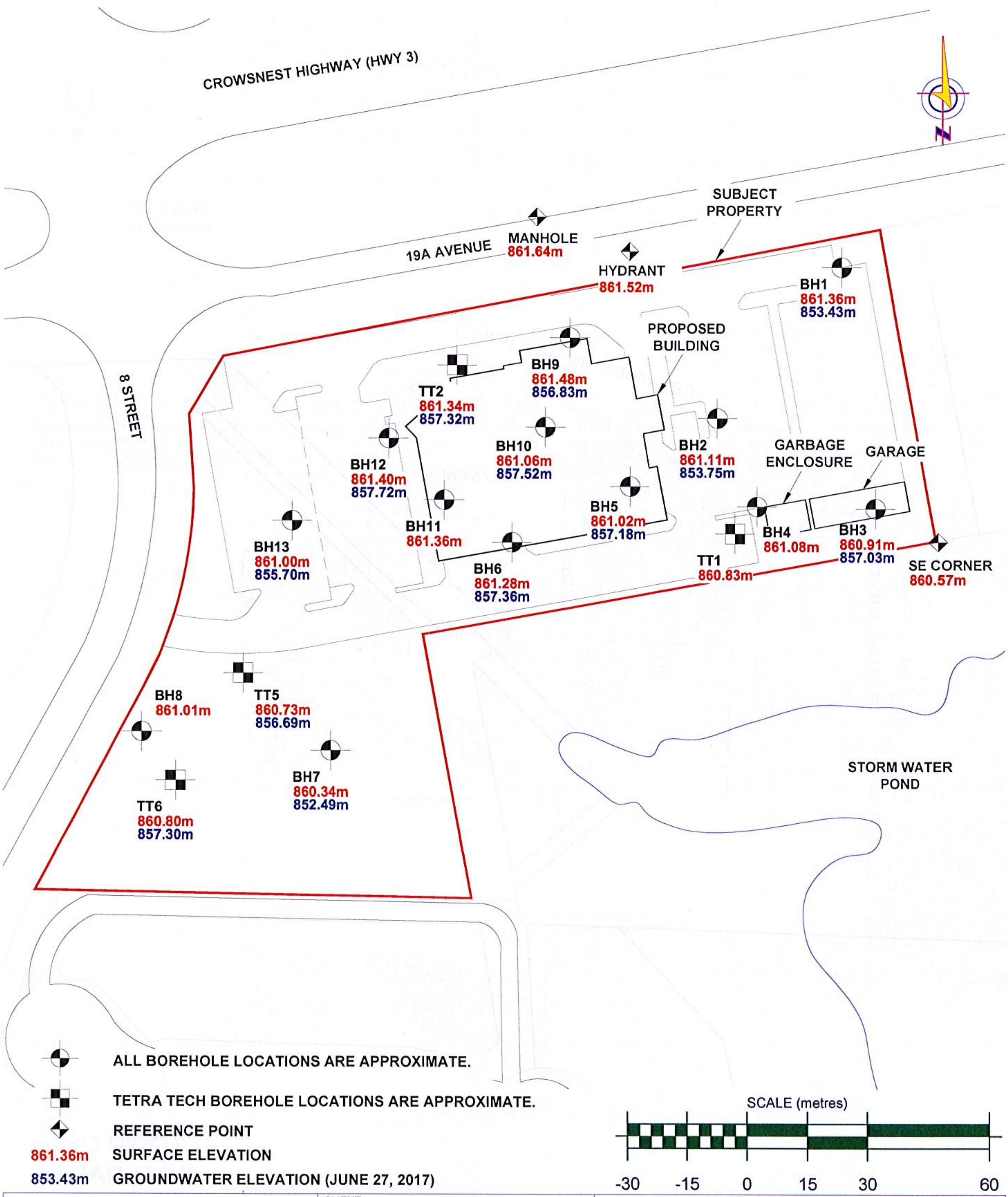
Parkland**GEO**

CLIENT:

KEY PLAN

PROPOSED RCMP BUILDING
LOT 93, BLOCK 1, PLAN 151 0788, COALDALE, ALBERTA

DRAWN: NC	CHK'D.: TB	REV #: 0	DATE: JULY 2017
SCALE: NTS	JOB NO. LE0114		DRAWING NO. FIGURE 1



⊕ ALL BOREHOLE LOCATIONS ARE APPROXIMATE.

▚ TETRA TECH BOREHOLE LOCATIONS ARE APPROXIMATE.

◆ REFERENCE POINT

861.36m SURFACE ELEVATION

853.43m GROUNDWATER ELEVATION (JUNE 27, 2017)

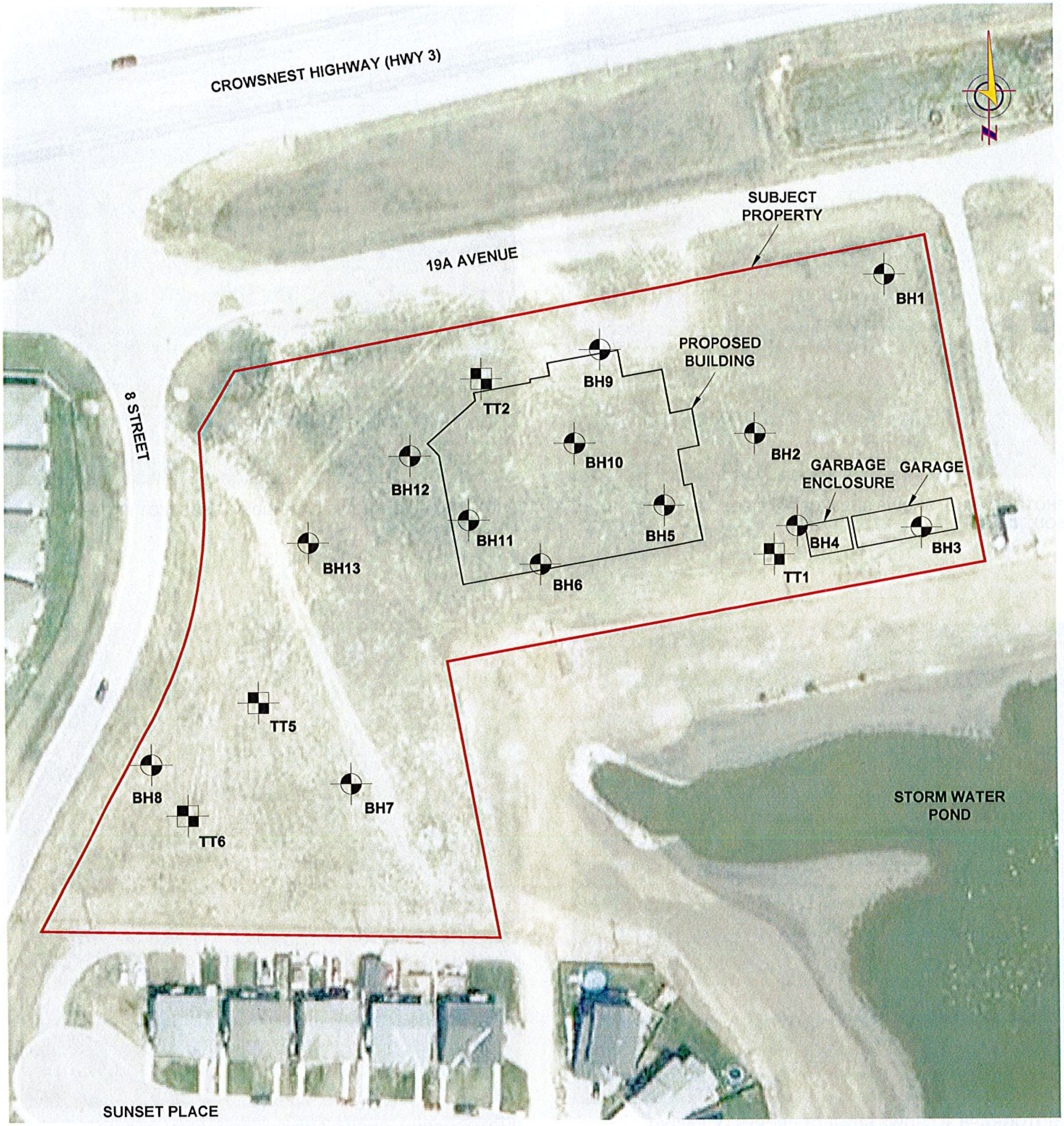
ParklandGEO

CLIENT:

SITE PLAN

PROPOSED RCMP BUILDING
LOT 93, BLOCK 1, PLAN 151 0788, COALDALE, ALBERTA

DRAWN: NC	CHK'D.: TB	REV #: 0	DATE: JULY 2017
SCALE: 1:1250	JOB NO. LE0114		DRAWING NO. FIGURE 2



NOTE: AERIAL PHOTOGRAPH OBTAINED FROM GOOGLE EARTH, , DATED JULY 17, 2015.

⊕ **ALL BOREHOLE LOCATIONS ARE APPROXIMATE.**

▚ **TETRA TECH BOREHOLE LOCATIONS ARE APPROXIMATE.**

SCALE (metres)

-30 -15 0 15 30 60

Parkland GEO

CLIENT:

AERIAL PLAN

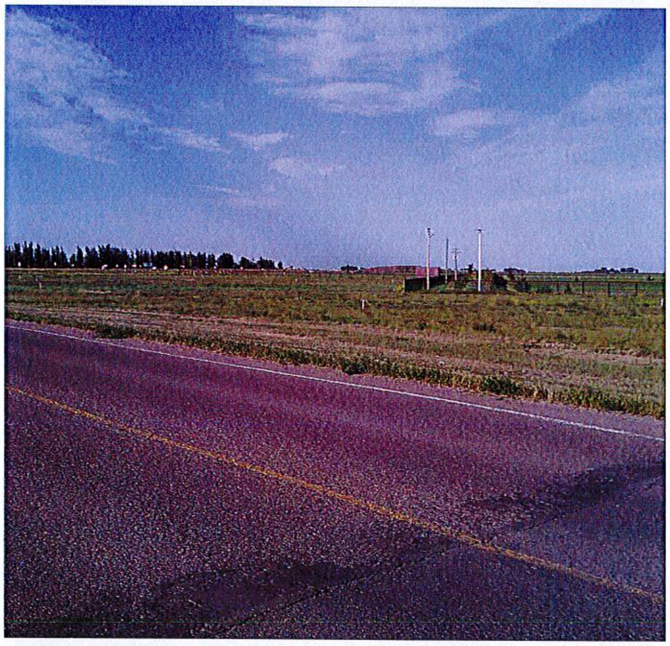
PROPOSED RCMP BUILDING
LOT 93, BLOCK 1, PLAN 151 0788, COALDALE, ALBERTA

DRAWN:	CHK'D.:	REV #:	DATE:
NC	TB	0	JULY 2017

SCALE:	JOB NO.	DRAWING NO.
1:1250	LE0114	FIGURE 3

PHOTOGRAPH 1: SHOWS SUBJECT PROPERTY, FACING SOUTHEAST

PHOTOGRAPH 2: SHOWS EAST SIDE OF SUBJECT PROPERTY, FACING SOUTH



PHOTOGRAPH 3: SHOWS SUBJECT PROPERTY, FACING NORTHEAST

PHOTOGRAPH 4: SHOWS SUBJECT PROPERTY, FACING SOUTHWEST

Parkland GEO

CLIENT:

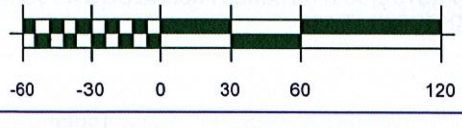
SITE PHOTOGRAPHS

PROPOSED RCMP BUILDING
LOT 93, BLOCK 1, PLAN 151 0788, COALDALE, ALBERTA

DRAWN:	CHK'D.:	REV #:	DATE:
NC	TB	0	JULY 2017

SCALE:	JOB NO.	DRAWING NO.
NTS	LE0114	FIGURE 4

AERIAL PHOTOGRAPH OBTAINED FROM ALBERTA SUSTAINABLE RESOURCES, DATED JULY 23, 1989.



ParklandGEO

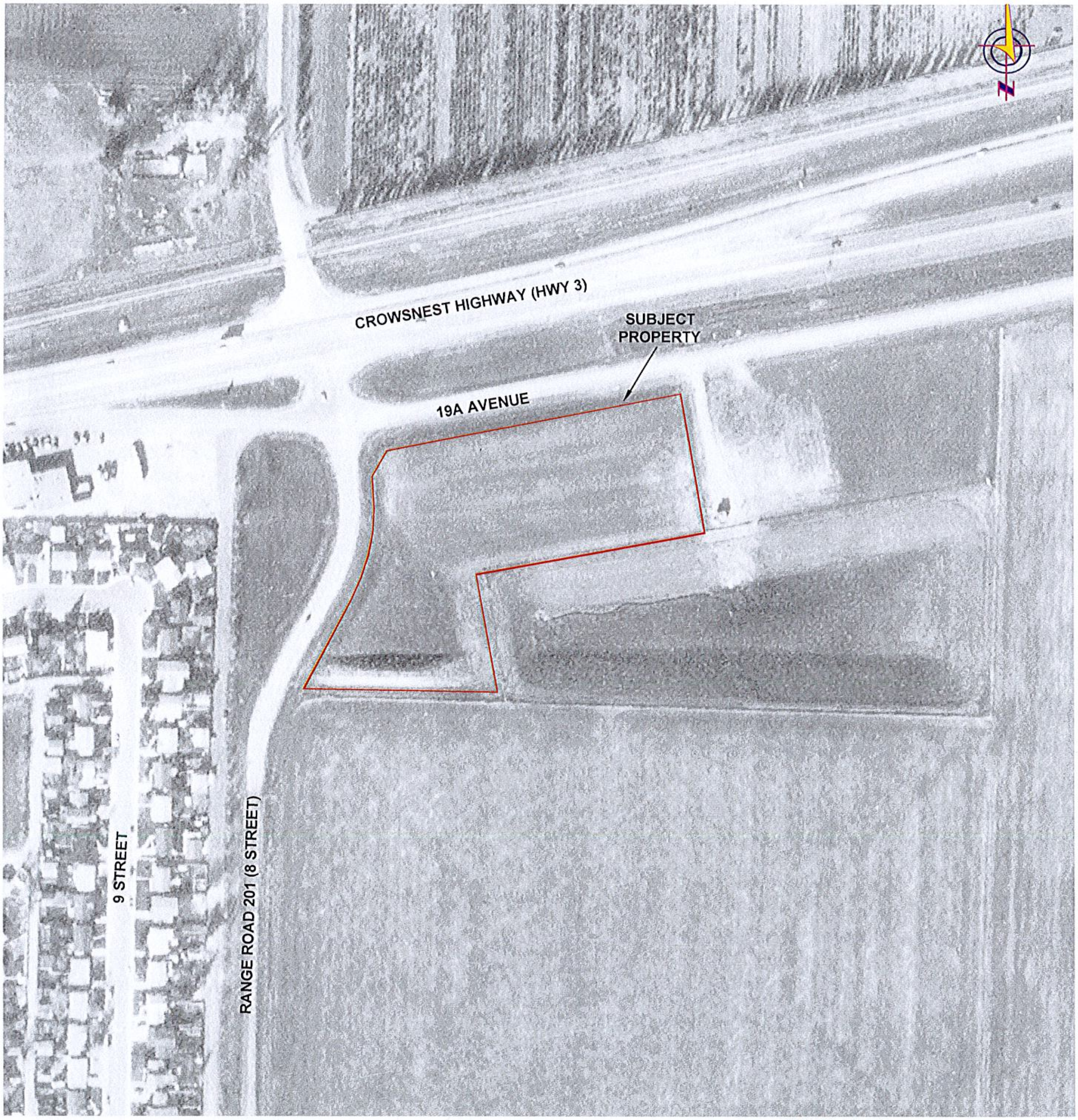
CLIENT:

1989 AERIAL PHOTOGRAPH

PROPOSED RCMP BUILDING
LOT 93, BLOCK 1, PLAN 151 0788, COALDALE, ALBERTA

DRAWN:	CHK'D.:	REV #:	DATE:
NC	TB	1	JULY 2017

SCALE:	JOB NO.	DRAWING NO.
1:3000	LE0114	FIGURE 5



AERIAL PHOTOGRAPH OBTAINED FROM ALBERTA SUSTAINABLE RESOURCES, DATED OCTOBER 17, 1996.

SCALE (metres)

-60 -30 0 30 60 120

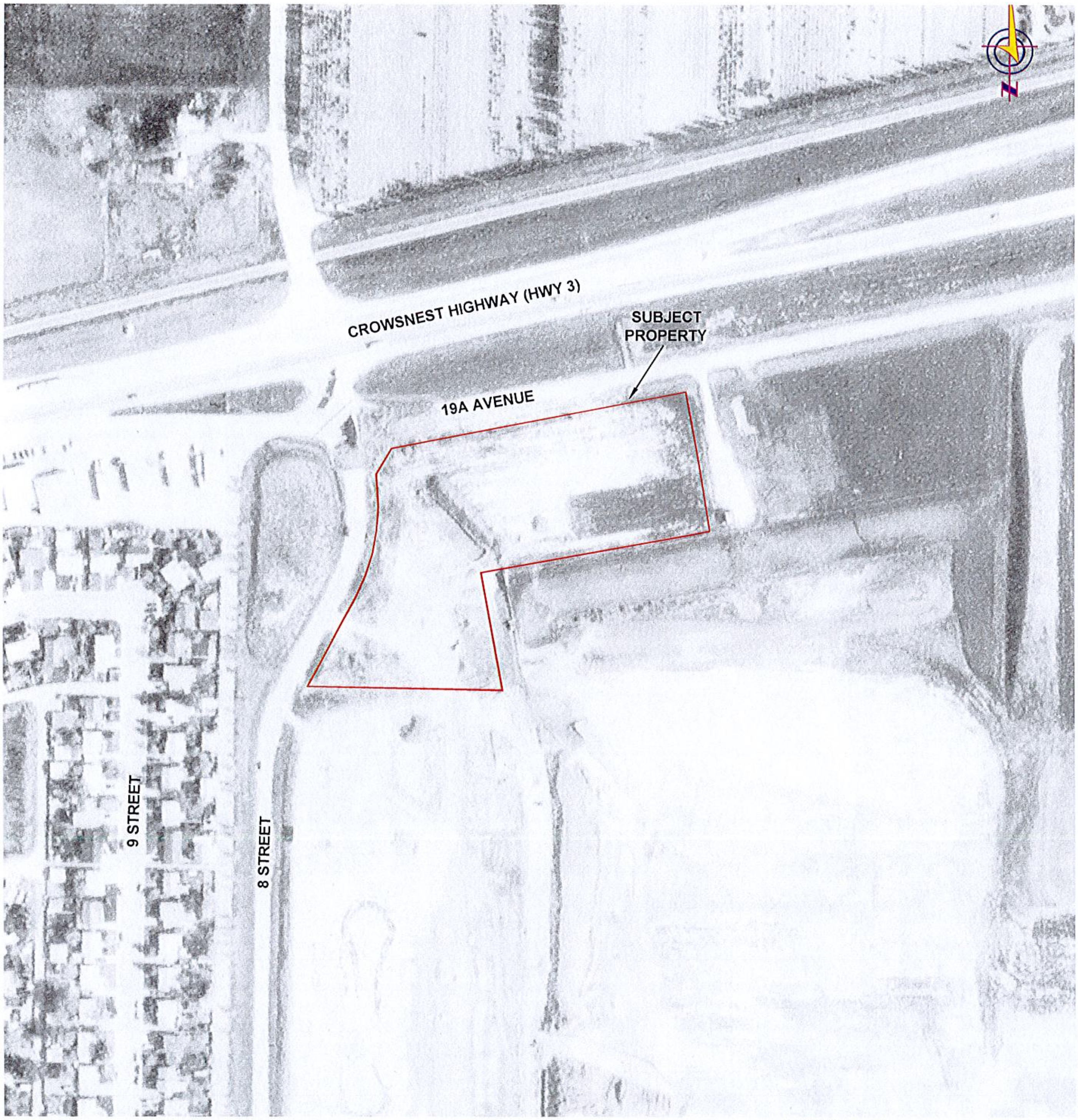
Parkland**GEO**

CLIENT:

1996 AERIAL PHOTOGRAPH

PROPOSED RCMP BUILDING
LOT 93, BLOCK 1, PLAN 151 0788, COALDALE, ALBERTA

DRAWN: NC	CHK'D.: TB	REV #: 1	DATE: JULY 2017
SCALE: 1:3000	JOB NO. LE0114		DRAWING NO. FIGURE 6

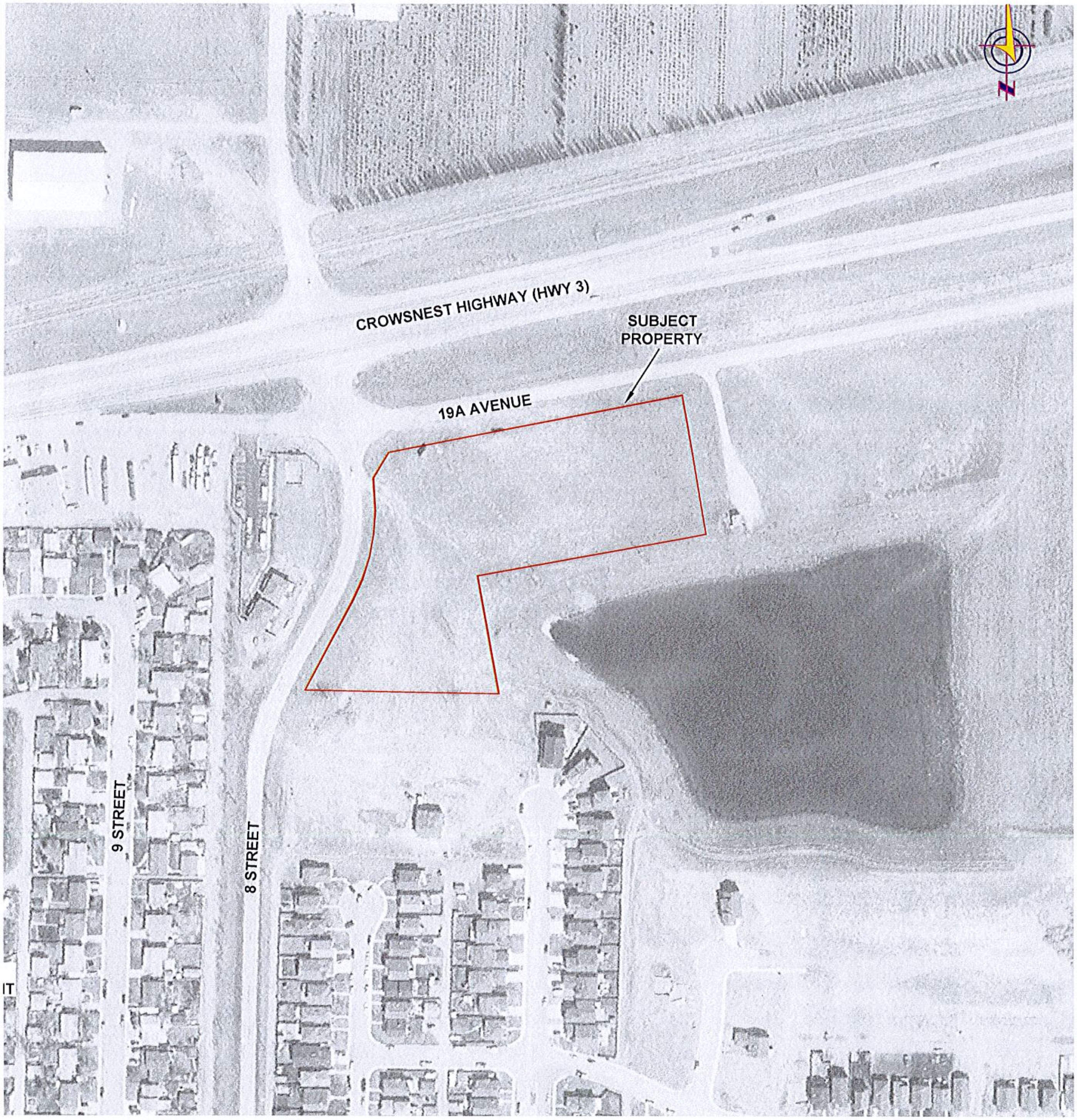


AERIAL PHOTOGRAPH OBTAINED FROM ALBERTA SUSTAINABLE RESOURCES, DATED MAY 24, 2003.

SCALE (metres)

-60 -30 0 30 60 120

ParklandGEO	CLIENT:	**2003 AERIAL PHOTOGRAPH**		
		PROPOSED RCMP BUILDING LOT 93, BLOCK 1, PLAN 151 0788, COALDALE, ALBERTA		
		DRAWN: NC	CHK'D.: TB	REV #: 1
		DATE: JULY 2017		
		SCALE: 1:3000	JOB NO. LE0114	DRAWING NO. FIGURE 7



AERIAL PHOTOGRAPH OBTAINED FROM ALBERTA SUSTAINABLE RESOURCES, DATED MAY 2, 2009.

SCALE (metres)

-60 -30 0 30 60 120

Parkland GEO

CLIENT:

2009 AERIAL PHOTOGRAPH

PROPOSED RCMP BUILDING
LOT 93, BLOCK 1, PLAN 151 0788, COALDALE, ALBERTA

DRAWN:	CHK'D.:	REV #:	DATE:
NC	TB	1	JULY 2017

SCALE:	JOB NO.	DRAWING NO.
1:3000	LE0114	FIGURE 8

APPENDIX A

Borehole Logs (14)
Explanation of Terms and Symbols
Frost Protection

Parkland**GEO**

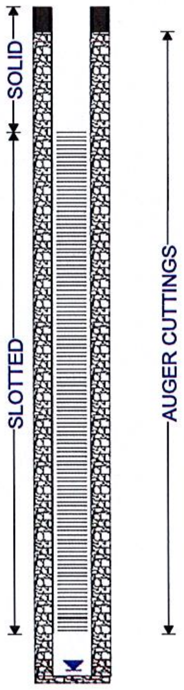
CLIENT: The RCMP
SITE: Coaldale RCMP Building
NOTES:

BOREHOLE NO.: BH 1
PROJECT NO.: LE0114
BH LOCATION:

SUBSURFACE PROFILE

Depth (m)	Description	Symbol	Moisture (Wp |-----X-----| Wl) 25 50 75	Type	Sample No	SPT (N)	Comments	Elevation (m)
	GROUND SURFACE							861.36
0	***Fill*** Clay, sillty, trace sand, trace gravel, stiff, low to medium plastic, coal inclusions and rust staining, brown, damp.	▨	14	G	1G1			860.36
1	***Clay*** Silty, trace sand, firm, medium plastic, brown, moist.	▨	21	▨	1D1	20	SO_4 = 1.82 %	
2			22				PP = 400 kPa	
3	***Till*** Clay, silty, trace sand, trace gravel, firm, medium plastic, coal inclusions and rust staining, brown, moist. - becoming wet, sand inclusions.	▨	17	▨	1D2	13		858.16
4			15	G	1G2			
5				▨	1D3	11		
6			15	▨	1D4	14	PP = 200 kPa	
7			18	G	1G3			
				▨	1D5	19		853.36
8	End of hole at 8.0 m. 25 mm PVC standpipe installed. Dry at completion. Backfilled with auger cuttings. Water level at 7.93 m on June 27, 2017.							
9								
10								
11								
12								
13								
14								
15								

Well Completion Details



LOGGED BY: TB
CONTRACTOR: Chilako Drilling
RIG/METHOD: Truck Mounted/Solid Stem
DATE: June 19, 2017
CALIBRATION:

GROUND ELEVATION: 861.36 m
NORTHING: 5510017.12 m
EASTING: 384872.64 m

ParklandGEO

CLIENT: The RCMP
SITE: Coaldale RCMP Building
NOTES:

BOREHOLE NO.: BH 2
PROJECT NO.: LE0114
BH LOCATION:

SUBSURFACE PROFILE

Depth (m)	Description	Moisture (Wp |-----X-----| Wl) 25 50 75	Type	Sample No	SPT (N)	Comments	Elevation (m)
0	GROUND SURFACE						861.11
0 – 0.8	***Fill*** Clay, silty, trace sand, trace gravel, stiff, low to medium plastic, coal inclusions and rust staining, brown, damp.	13					860.31
0.8 – 3.0	***Clay*** Silty, trace sand, firm,medium plastic, brown, moist.	16 17		2D1	19		858.11
3.0 – 8.0	***Till*** Clay, silty, trace sand, trace gravel, firm, medium plastic, coal inclusions and rust staining, brown, moist. - becoming wet, sand inclusions.	23 16 16 16		2D2 2G1 2D3 2D4 2D5	11 12 18 24	PP = 300 kPa Grain Size Analysis: Gravel = 0 % Sand = 6.8 % Silt = 38.3 % Clay = 54.9 % PP = 350 kPa	853.11
8	End of hole at 8.0 m. 25 mm PVC standpipe installed. Dry at completion. Backfilled with auger cuttings. Water level at 7.36 m on June 27, 2017.						

Well Completion Details

LOGGED BY: TB
CONTRACTOR: Chilako Drilling
RIG/METHOD: Truck Mounted/Solid Stem
DATE: June 19, 2017
CALIBRATION:

GROUND ELEVATION: 861.11 m
NORTHING: 5509979.64 m
EASTING: 384841.76 m

ParklandGEO

CLIENT: The RCMP
SITE: Coaldale RCMP Building
NOTES:

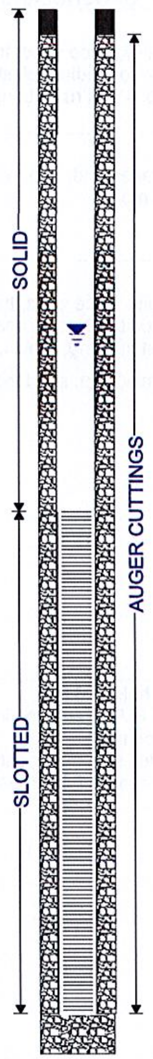
BOREHOLE NO.: BH 3

PROJECT NO.: LE0114
BH LOCATION:

SUBSURFACE PROFILE

Depth (m)	Description	Moisture (Wp |-----X-----| Wl) 25 50 75	Type	Sample No	SPT (N)	Comments	Elevation (m)
	GROUND SURFACE						860.91
0	***Fill*** Clay, siltly, trace sand, trace gravel, stiff, low to medium plastic, coal inclusions and rust staining, brown, damp.						860.31
1		12 ●					
2	***Clay*** Silty, trace sand, firm, medium plastic, damp to moist.	16 ●	▨	3D1	23		
3		15 ●	▨	3D2	21		857.41
4	***Till*** Clay, silty, trace sand, trace gravel, firm, medium plastic, coal inclusions and rust staining, brown, moist.	24 ●				PP = 250 kPa	
5		26 ●	▨	3D3	14		
6		20 ●	▨	3D4	13	PP = 250 kPa	
7		16 ●					
8		18 ●	▨	3D5	17	PP = 300 kPa	
9	- becoming wet, free water.	19 ●	▨	3D6	19		851.91
10		17 ●					
11		14 ●	▨	3D7	28		
12		16 ●	▨	3D8	24	PP = 400 kPa	848.41
13	End of hole at 12.5 m. 25 mm PVC standpipe installed. Wet at completion. Backfilled with auger cuttings. Water level at 3.88 m on June 27, 2017.						
14							
15							

Well Completion Details



LOGGED BY: TB
CONTRACTOR: Chilako Drilling
RIG/METHOD: Truck Mounted/Solid Stem
DATE: June 19, 2017
CALIBRATION:

GROUND ELEVATION: 860.91 m
NORTHING: 5509957.33 m
EASTING: 384881.22 m

Parkland GEO

CLIENT: The RCMP
SITE: Coaldale RCMP Building
NOTES:

BOREHOLE NO.: BH 4

PROJECT NO.: LE0114
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								861.08
	Fill Clay, sillty, trace sand, trace gravel, stiff, low to medium plastic, coal inclusions and rust staining, brown, damp.		14	G	4G1				
	Clay Silty, trace sand, firm, medium plastic, brown, moist.		17 17	▨	4D1	22			860.28
	Till Clay, silty, trace sandy, trace gravel, firm, high plastic, coal inclsuions and rust stains, brown, moist to wet.		17 18 19 18 19 23 16 15 14	▨ ▨ ▨ ▨ ▨ ▨	4D2 4D3 4D4 4D5 4D6 4D7	15 10 11 14 33 27	PP = 300 kPa		857.98
12	End of hole at 12.0 m. Wet at completion. Water level at 11.3 m bg Backfilled with auger cuttings.								849.08

LOGGED BY: TB
CONTRACTOR: Chilako Drilling
RIG/METHOD: Truck Mounted/Solid Stem
DATE: June 19, 2017
CALIBRATION:

GROUND ELEVATION: 861.08 m
NORTHING: 5509957.78 m
EASTING: 384851.73 m

PAGE 1 of 1

ParklandGEO

CLIENT: The RCMP
SITE: Coaldale RCMP Building
NOTES:

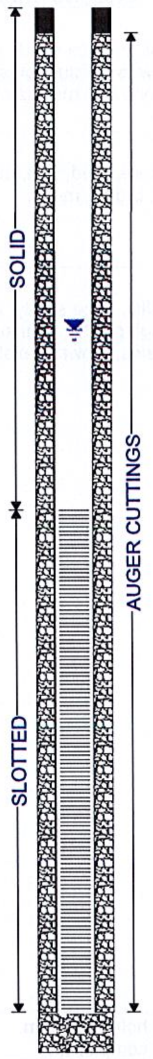
BOREHOLE NO.: BH 5

PROJECT NO.: LE0114
BH LOCATION:

SUBSURFACE PROFILE

Depth (m)	Description	Moisture (Wp |-----X-----| Wl) 25 50 75	Type	Sample No	SPT (N)	Comments	Elevation (m)
	GROUND SURFACE						861.23
0	***Fill*** Clay, silty, trace sand, trace gravel, stiff, low to medium plastic, coal inclusions and rust staining, brown, damp.						860.53
1		16				PP = 400 kPa	
2	***Clay*** Silty, trace sand, trace gravel, firm, medium plastic, brown, moist.	26	▨	5D1	14		
3		20					858.03
4	***Till*** Clay, silty, trace sandy, trace gravel, firm, high plastic, coal inclsuions and rust stains, brown, moist to wet.	17	▨ G	5D2 5G1	7		
5		19	▨	5D3	7		
6		22	▨	5D4	10		
7		15					
8		19					
9		18	▨	5D5	15		
10		16					
11		17					
12		17					849.23
	Sand Silty, clayey, compact, poorly graded, brown, wet.		▨	5D6	18	Grain Size Analysis: Gravel = 0.5 % Sand = 69.8 % Silt and Clay = 29.7 %	848.73
13	End of hole at 12.5 m. 25 mm PVC standpipe installed. Wet at completion. Backfilled with auger cuttings. Water level at 3.84 m on June 27, 2017.						
14							
15							

Well Completion Details



LOGGED BY: TB
CONTRACTOR: Chilako Drilling
RIG/METHOD: Truck Mounted/Solid Stem
DATE: June 19, 2017
CALIBRATION:

GROUND ELEVATION: 861.23 m
NORTHING: 5509962.76 m
EASTING: 384819.74 m

ParklandGEO

CLIENT: The RCMP
SITE: Coaldale RCMP Building
NOTES:

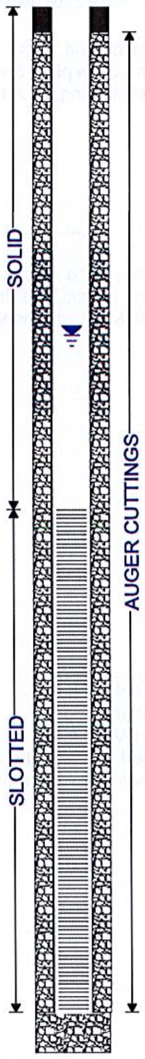
BOREHOLE NO.: BH 6

PROJECT NO.: LE0114
BH LOCATION:

SUBSURFACE PROFILE

Depth (m)	Description	Symbol	Moisture (Wp |-----X-----| Wl) 25 50 75	Type	Sample No	SPT (N)	Comments	Elevation (m)
0	GROUND SURFACE							861.28
	Fill Clay, silty, trace sand, trace gravel, stiff, low to medium plastic, coal inclusions and rust staining, brown, damp.		15					860.38
	Clay Silty, trace sand, medium plastic, firm, brown, moist.		9 23		6D1	19		858.18
	Till Clay, silty, trace sandy, trace gravel, firm, high plastic, coal inclsuions and rust stains, brown, moist to wet. - free water at 10.5 m.		9 18 21 22 19 19 17 16 16		6D2 6D3 6D4 6D5 6D6	24 18 17 21 26	PP = 300 kPa	850.78 848.78
15	End of hole at 12.5 m. Dry at completion. 25 mm PVC standpipe installed. Backfilled with auger cuttings. Water level at 3.92 m on June 27, 2017.							

Well Completion Details



LOGGED BY: TB
CONTRACTOR: Chilako Drilling
RIG/METHOD: Truck Mounted/Solid Stem
DATE: June 19, 2017
CALIBRATION:

GROUND ELEVATION: 861.28 m
NORTHING: 5509948.91 m
EASTING: 38490.56 m

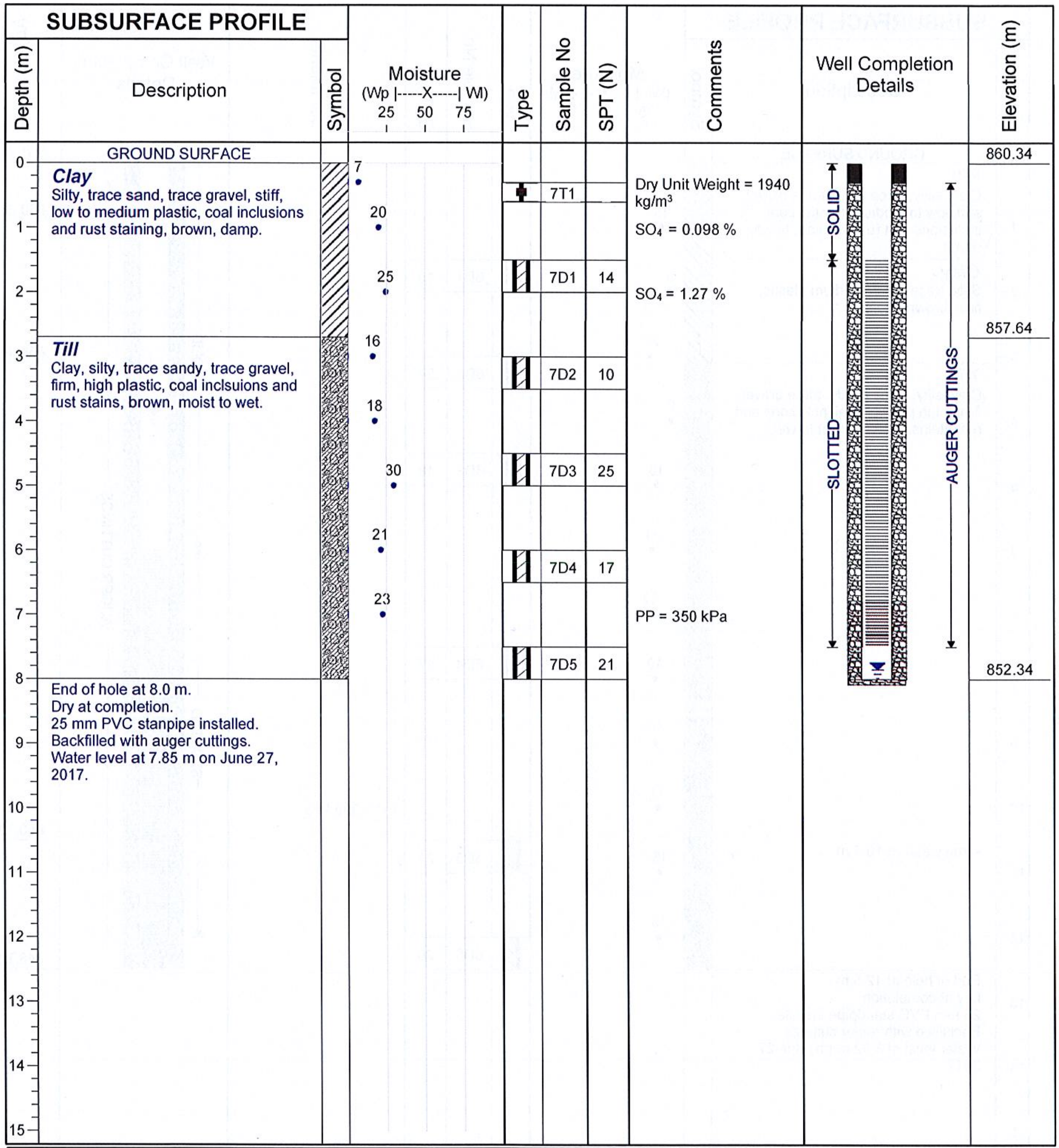
BOREHOLE NO.: BH 7

ParklandGEO

CLIENT: The RCMP
SITE: Coaldale RCMP Building
NOTES:

PROJECT NO.: LE0114
BH LOCATION:

SUBSURFACE PROFILE



GROUND SURFACE

Clay
Silty, trace sand, trace gravel, stiff, low to medium plastic, coal inclusions and rust staining, brown, damp.

Till
Clay, silty, trace sandy, trace gravel, firm, high plastic, coal inclsuions and rust stains, brown, moist to wet.

End of hole at 8.0 m.
Dry at completion.
25 mm PVC stanpipe installed.
Backfilled with auger cuttings.
Water level at 7.85 m on June 27, 2017.

Sample No	SPT (N)	Comments
7T1		Dry Unit Weight = 1940 kg/m^3 SO_4 = 0.098 %
7D1	14	SO_4 = 1.27 %
7D2	10	
7D3	25	
7D4	17	PP = 350 kPa
7D5	21	

Elevation (m): 860.34, 857.64, 852.34

LOGGED BY: TB
CONTRACTOR: Chilako Drilling
RIG/METHOD: Truck Mounted/Solid Stem
DATE: June 19, 2017
CALIBRATION:

GROUND ELEVATION: 860.34 m
NORTHING: 5509897.19 m
EASTING: 38745.43 m

ParklandGEO

CLIENT: The RCMP
SITE: Coaldale RCMP Building
NOTES:

BOREHOLE NO.: BH 8
PROJECT NO.: LE0114
BH LOCATION:

SUBSURFACE PROFILE

Depth (m)	Description	Symbol	Elevation (m)
0	GROUND SURFACE		861.01
0	***Fill*** Clay, silty, trace sand, trace gravel, stiff, low to edium plastic, coal inclusions and rust staining, brown, damp.	▩	860.41
0.6	***Clay*** Sillty, trace sand, trace gravel, stiff, low to medium plastic, coal inclusions and rust staining, brown, damp.	▨	858.01
3	***Till*** Clay, silty, trace sandy, trace gravel, firm, high plastic, coal inclsuions and rust stains, brown, moist to wet.	▦	853.01
8	End of hole at 8.0 m. Dry at completion. Backfilled with auger cuttings.		

Depth (m)	Moisture (Wp |-----X-----| Wl)	Type	Sample No	SPT (N)	Comments	Well Completion Details
1	13					
1.75		▨	8D1	26		
2	13					
3	27					
3.25		▨	8D2	12		
4	16					
4.75		▨	8D3	14		
5	21					
6	31				PP = 200 kPa	
6.25		▨	8D4	14		
6.75		G	8G1			
7	18					
7.75		▨	8D5	22		

LOGGED BY: TB
CONTRACTOR: Chilako Drilling
RIG/METHOD: Truck Mounted/Solid Stem
DATE: June 19, 2017
CALIBRATION:

GROUND ELEVATION: 861.01 m
NORTHING: 5509901.84 m
EASTING: 384698.52 m

ParklandGEO

CLIENT: The RCMP
SITE: Coaldale RCMP Building
NOTES:

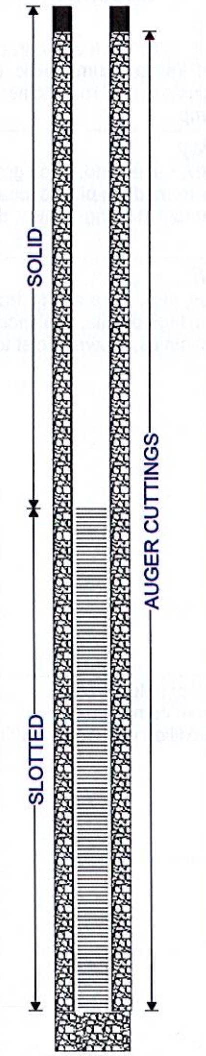
BOREHOLE NO.: BH 9

PROJECT NO.: LE0114
BH LOCATION:

SUBSURFACE PROFILE

Depth (m)	Description	Symbol	Moisture (Wp |-----X-----| Wl) 25 50 75	Type	Sample No	SPT (N)	Comments	Elevation (m)
0	GROUND SURFACE							861.48
1	***Fill*** Clay, sillty, trace sand, trace gravel, stiff, low to medium plastic, coal inclusiions and rust staining, brown, damp.		15				PP = 400 kPa +	860.38
2	***Clay*** Silty, trace sand, firm, medium plastic, brown, moist.		15	▨	9D1	20		
3	***Till*** Clay, silty, trace sandy, trace gravel, firm, high plastic, coal inclsuions and rust stains, brown, moist to wet.		25	▨	9D2	13		858.58
4			30				PP = 150 kPa	
5			25	▨	9D3	11		
6			21					
7			17					
8			24	▨	9D4	11		
9			20				PP = 300 kPa	
10			17				PP = 200 kPa	850.98
11	- free water at 10.6 mbg		16	▨	9D5	20		
12			16	▨	9D6	28	PP = 300 kPa	848.98
13	End of hole at 12.5 m. Wet at completion. 25 mm PVC standpipe installed. Backfilled with auger cuttings.							
14								
15								

Well Completion Details



LOGGED BY: TB
CONTRACTOR: Chilako Drilling
RIG/METHOD: Truck Mounted/Solid Stem
DATE: June 20, 2017
CALIBRATION:

GROUND ELEVATION: 861.48 m
NORTHING: 5509999.58 m
EASTING: 384804.79 m

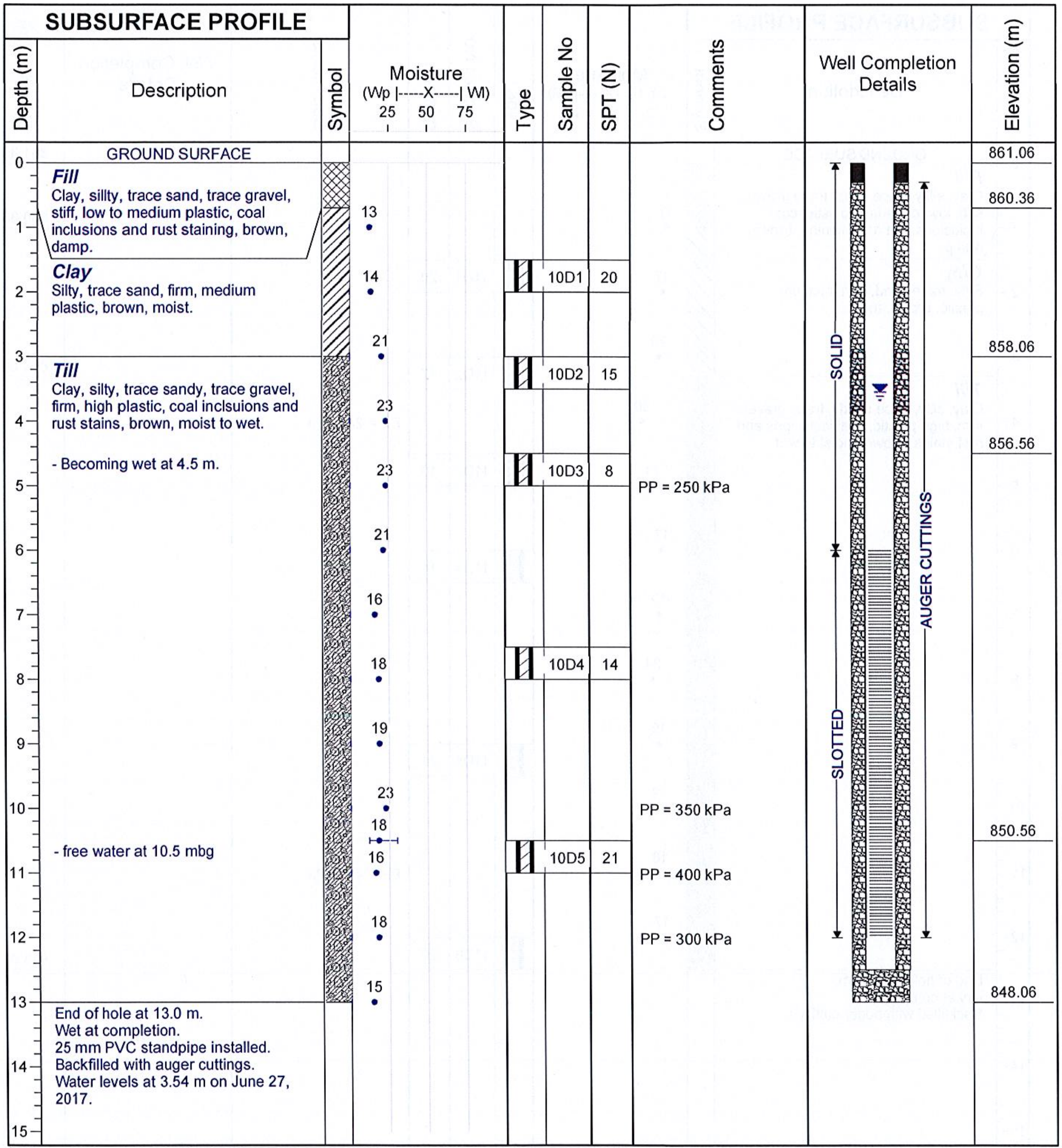
ParklandGEO

CLIENT: The RCMP
SITE: Coaldale RCMP Building
NOTES:

BOREHOLE NO.: BH 10

PROJECT NO.: LE0114
BH LOCATION:

SUBSURFACE PROFILE



GROUND SURFACE

Fill
Clay, silty, trace sand, trace gravel, stiff, low to medium plastic, coal inclusions and rust staining, brown, damp.

Clay
Silty, trace sand, firm, medium plastic, brown, moist.

Till
Clay, silty, trace sandy, trace gravel, firm, high plastic, coal inclsuions and rust stains, brown, moist to wet.

- Becoming wet at 4.5 m.

- free water at 10.5 mbg

End of hole at 13.0 m.
Wet at completion.
25 mm PVC standpipe installed.
Backfilled with auger cuttings.
Water levels at 3.54 m on June 27, 2017.

Depth (m)	Moisture	Sample No	SPT (N)	Comments
1	13			
2	14	10D1	20	
3	21			
3.5		10D2	15	
4	23			
5	23	10D3	8	PP = 250 kPa
6	21			
7	16			
8	18	10D4	14	
9	19			
10	23			PP = 350 kPa
10.5	18			
11	16	10D5	21	PP = 400 kPa
12	18			PP = 300 kPa
13	15			

Elevation (m)
861.06
860.36
858.06
856.56
850.56
848.06

LOGGED BY: TB
CONTRACTOR: Chilako Drilling
RIG/METHOD: Truck Mounted/Solid Stem
DATE: June 20, 2017
CALIBRATION:

GROUND ELEVATION: 861.06 m
NORTHING: 5509977.44 m
EASTING: 384798.75 m

Parkland**GEO**

CLIENT: The RCMP
SITE: Coaldale RCMP Building
NOTES:

BOREHOLE NO.: BH 11

PROJECT NO.: LE0114
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								861.36
0	***Fill*** Clay, sillty, trace sand, trace gravel, stiff, low to medium plastic, coal inclusions and rust staining, brown, damp.								
1			13						860.36
	Clay Silty, trace sand, firm, medium plastic, brown, moist.			▨	11D1	20			
2			17						
3			20						
	Till Clay, silty, trace sandy, trace gravel, firm, high plastic, coal inclsuions and rust stains, brown, moist to wet.			▨	11D2	17			858.06
4			30				PP = 200 kPa		
				▨	11D3	10			
5			24						
6			17						
				▨	11D4	14			
7			19						
8			24						
9			19						
				▨	11D5	21			
10			19						
11			18				PP = 400 kPa		
12			17						
				▨	11D6	20			848.86
13	End of hole at 12.5 m. Dry at completion. Backfilled with auger cuttings.								
14									
15									

LOGGED BY: TB
CONTRACTOR: Chilako Drilling
RIG/METHOD: Truck Mounted/Solid Stem
DATE: June 20, 2017
CALIBRATION:

GROUND ELEVATION: 861.36 m
NORTHING: 5509959.36 m
EASTING: 384773.71 m

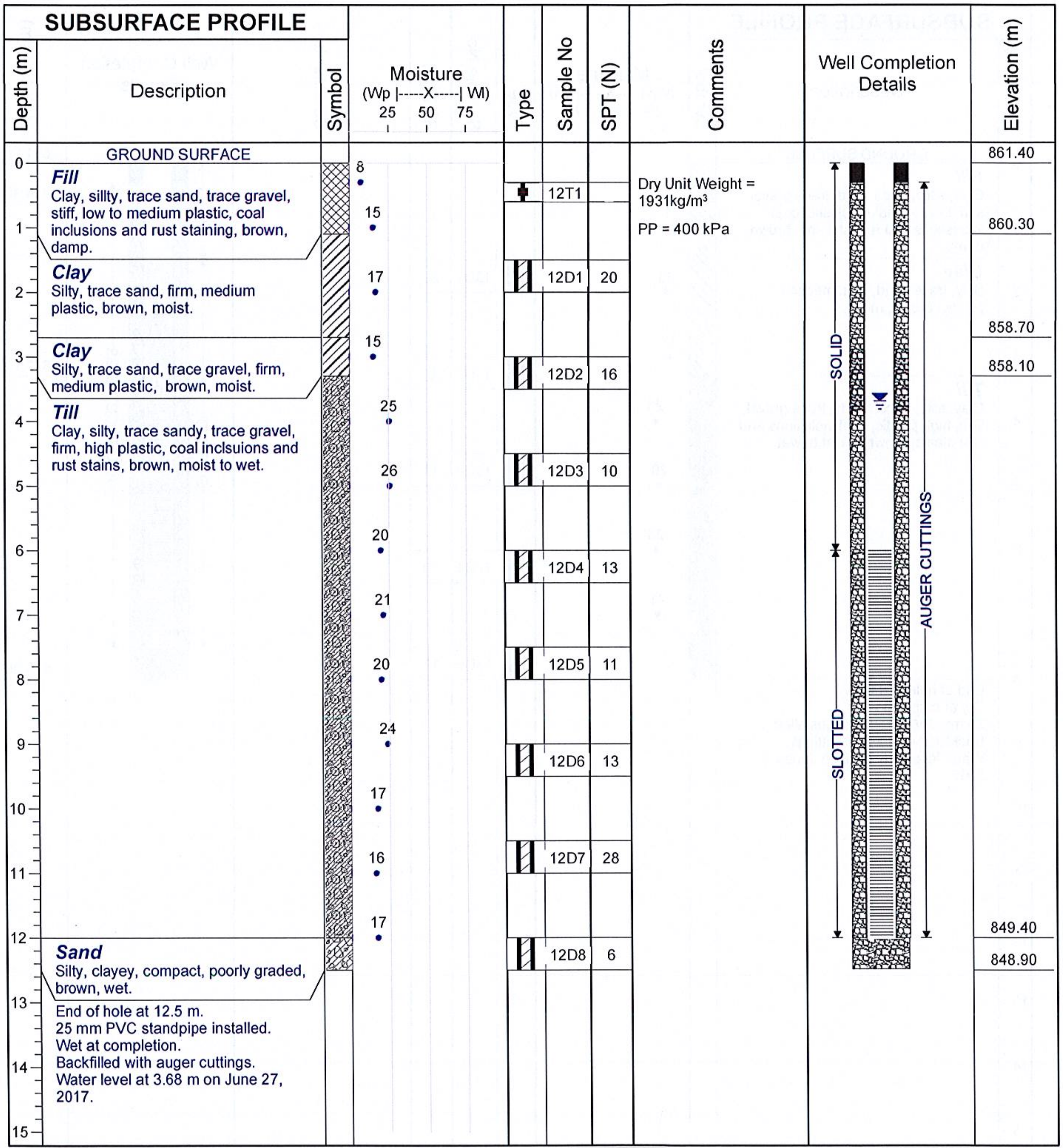
ParklandGEO

CLIENT: The RCMP
SITE: Coaldale RCMP Building
NOTES:

BOREHOLE NO.: BH 12

PROJECT NO.: LE0114
BH LOCATION:

SUBSURFACE PROFILE



GROUND SURFACE

Fill
Clay, silty, trace sand, trace gravel, stiff, low to medium plastic, coal inclusions and rust staining, brown, damp.

Clay
Silty, trace sand, firm, medium plastic, brown, moist.

Clay
Silty, trace sand, trace gravel, firm, medium plastic, brown, moist.

Till
Clay, silty, trace sandy, trace gravel, firm, high plastic, coal inclsuions and rust stains, brown, moist to wet.

Sand
Silty, clayey, compact, poorly graded, brown, wet.

End of hole at 12.5 m.
25 mm PVC standpipe installed.
Wet at completion.
Backfilled with auger cuttings.
Water level at 3.68 m on June 27, 2017.

Dry Unit Weight = 1931kg/m³

PP = 400 kPa

LOGGED BY: TB
CONTRACTOR: Chilako Drilling
RIG/METHOD: Truck Mounted/Solid Stem
DATE: June 20, 2017
CALIBRATION:

GROUND ELEVATION: 861.40 m
NORTHING: 5509974.61 m
EASTING: 384759.92 m

ParklandGEO

CLIENT: The RCMP
SITE: Coaldale RCMP Building
NOTES:

BOREHOLE NO.: BH 13

PROJECT NO.: LE0114
BH LOCATION:

SUBSURFACE PROFILE

Depth (m)	Description	Elevation (m)
0	GROUND SURFACE	861.00
0	***Fill*** Clay, sillty, trace sand, trace gravel, stiff, low to medium plastic, coal inclusiions and rust staining, brown, damp.	
0.6	***Clay*** Silty, trace sand, firm, medium plastic, brown, moist.	860.40
3.3	***Till*** Clay, silty, trace sandy, trace gravel, firm, high plastic, coal inclsuions and rust stains, brown, moist to wet.	857.70
8.0	End of hole at 8.0 m. Dry at completion. 25 mm PVC standpipe installed. Backfilled with auger cuttings. Water level at 5.30 m on June 27, 2017.	853.00

Moisture (Wp |-----X-----| Wl) 25 50 75

Depth (m)	Moisture
1	11
2	15
3	13
4	23
5	20
6	23
7	21

Type	Sample No	SPT (N)	Comments
▨	13D1	29	
▨	13D2	9	
▨	13D3	11	
▨	13D4	10	
▨	13D5	13	

Well Completion Details

LOGGED BY: TB
CONTRACTOR: Chilako Drilling
RIG/METHOD: Truck Mounted/Solid Stem
DATE: June 20, 2017
CALIBRATION:

GROUND ELEVATION: 861.00 m
NORTHING: 5509954.18 m
EASTING: 384735.85 m

ParklandGEO

CLIENT: The RCMP
SITE: Coaldale RCMP Building
NOTES:

BOREHOLE NO.: BH 14

PROJECT NO.: LE0114
BH LOCATION:

SUBSURFACE PROFILE

<table>
<tr><th>Depth (m)</th><th>Description</th><th>Symbol</th><th>Moisture (Wp |-----X-----| Wl) 25 50 75</th><th>Type</th><th>Sample No</th><th>SPT (N)</th><th>Comments</th><th>Well Completion Details</th><th>Elevation (m)</th></tr>
<tr><td></td><td>GROUND SURFACE</td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td>861.00</td></tr>
<tr><td>0</td><td><i>Fill</i>
Clay, sillty, trace sand, trace gravel, stiff, low to medium plastic, coal inclusiions and rust staining, brown, damp.</td><td>▨</td><td>6
●</td><td>■</td><td>14T1</td><td></td><td>Dry Unit Weight = 1978 kg/m^3</td><td></td><td>860.40</td></tr>
<tr><td>1
2
3
4
5
6
7
8
9
10
11
12
13
14
15</td><td>End of hole at 0.6 m.
Dry at completion.
Backfilled with auger cuttings.
Shelby Tube only.</td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr>
</table>

LOGGED BY: TB
CONTRACTOR: Chilako Drilling
RIG/METHOD: Truck Mounted/Solid Stem
DATE: June 30, 2017
CALIBRATION:

GROUND ELEVATION: 861.00 m
NORTHING: 5509967 m
EASTING: 384820 m

THE PARKLANDGEO CONSULTING GROUP
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 materials, boundaries, and conditions have been established only at the borehole location 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 the borehole logs.

1. **PRINCIPAL SOIL TYPE** – The major soil type by weight of material or by behaviour.

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	0.075 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

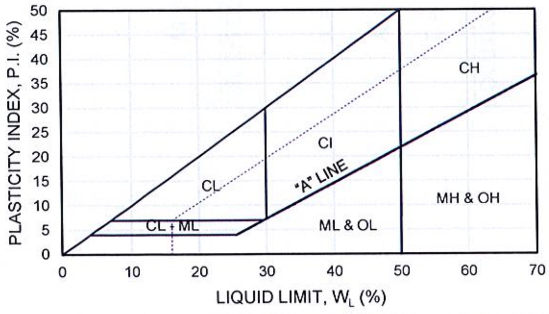
4. **CONSISTENCY OF FINE GRAINED SOILS** – The following terms are used relative to undrained shear strength and Standard Penetration Test (SPT), ASTM D1586, N value for blows per 300 mm. It is noted that this correlation needs to be used with caution as the correlation is only very approximate.

Description	Undrained Shear Strength, C_u (kPa)	N Value
Very Soft	Less than 12	Less than 2
Soft	12 to 25	2 to 4
Firm	25 to 50	4 to 8
Stiff	50 to 100	8 to 15
Very Stiff	100 to 150	15 to 30
Hard	Over 150	Over 30

THE PARKLANDGEO CONSULTING GROUP
EXPLANATION OF TERMS AND SYMBOLS

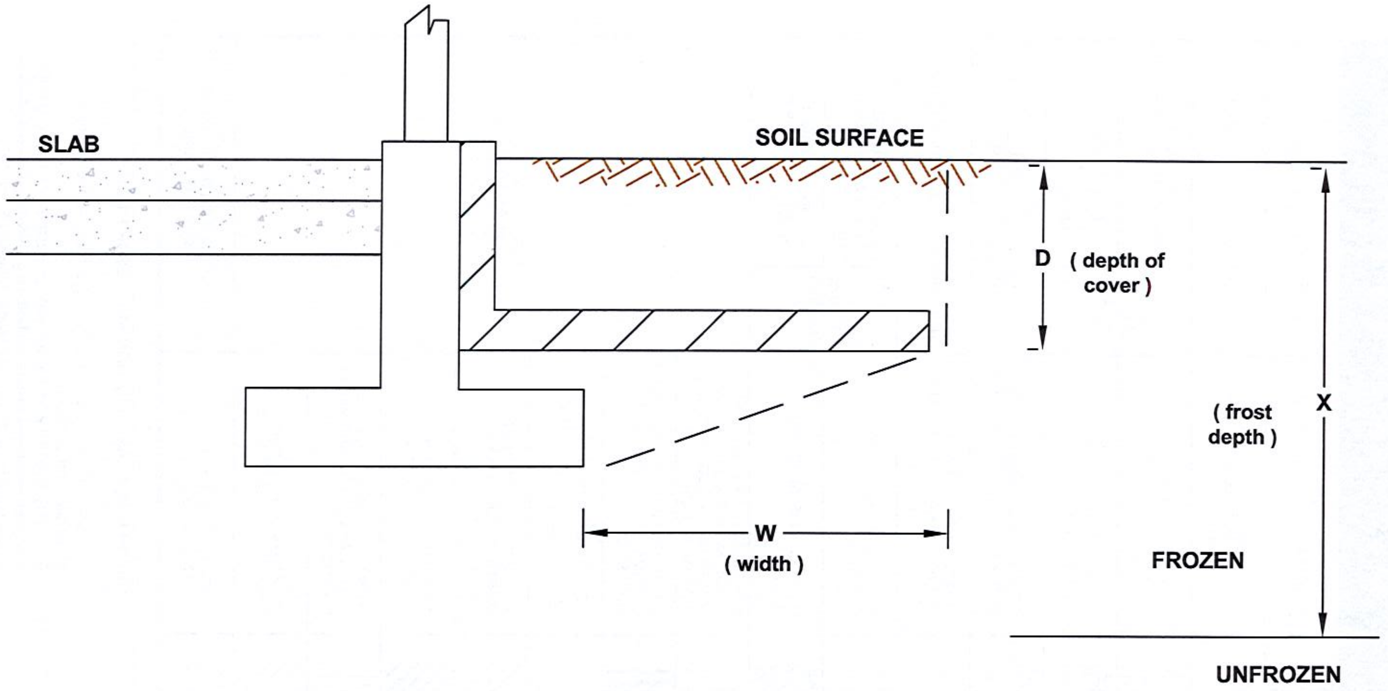
MODIFIED UNIFIED CLASSIFICATION SYSTEM FOR SOILS

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, GRAVEL-SAND MIXTURE, LITTLE OR NO FINES	$C_U = \frac{D_{60}}{D_{10}} > C_C = \frac{(D_{30})^2}{D_{10} \times D_{60}} = 1$ to 3	
			GP		POORLY GRADED GRAVELS, GRAVEL-SAND MIXTURES, LITTLE OR NO FINES	NOT MEETING 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. LESS THAN 7
	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	$C_U = \frac{D_{60}}{D_{10}} > C_C = \frac{(D_{30})^2}{D_{10} \times D_{60}} = 1$ to 3	
			SP		POORLY GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES	NOT MEETING 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. LESS THAN 7
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 FLOUR, SILTY OR CLAYEY FINE SANDS OR CLAYEY SILTS WITH SLIGHT PLASTICITY	CLASSIFICATION IS BASED UPON PLASTICITY CHART (SEE BELOW)	
		$W_L > 50\%$	**MH**		INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS, FINE SANDY OR SILTY SOILS		
	CLAYS ABOVE "A" LINE NEGLIGIBLE ORGANIC CONTENT	**$W_L < 30\%$**	**CL**		INORGANIC CLAYS OF LOW PLASTICITY, GRAVELLY, SANDY, OR SILTY SOILS		
		$30\% < W_L < 50\%$	**CI**		INORGANIC CLAYS OF MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS		
		$W_L > 50\%$	**CH**		INORGANIC CLAYS OF HIGH PLASTICITY, FAT CLAYS		
	ORGANIC SILTS & CLAYS BELOW "A" LINE	**$W_L < 50\%$**	**OL**		ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW AND MEDIUM PLASTICITY		
		$W_L > 50\%$	**OH**		ORGANIC CLAYS OF HIGH PLASTICITY, ORGANIC SILTS		
HIGHLY ORGANIC SOILS			**Pt**		PEAT AND OTHER HIGHLY ORGANIC SOILS	STRONG COLOR OR ODOR, AND OFTEN FIBROUS TEXTURE	



NOTES ON SOIL CLASSIFICATION AND DESCRIPTION:

1. Soil are classified and described according to their engineering properties and behaviour.
2. Boundary classification for soil with characteristics of two groups are given combined group symbols (e.g. GW-GC is a well graded gravel sand mixture with clay binder between 5 and 12%).
3. Soil classification is in accordance with the Unified Soil Classification System (ASTM D2487) with the exception that an inorganic clay of medium plasticity (CI) is recognized.
4. The use of modifying adjectives may be employed to define the estimated percentage range by eight of minor components.



FOOTING

D = depth of soil cover

W = projected width from foundation base

X = frost depth beneath uninsulated soil surface

WIDTH CRITERION

$W \geq X - D$

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CLIENT:

INSULATION CONFIGURATION FOR FROST PROTECTION

COALDALE RCMP BUILDING

DRAWN: MMc	CHK'D.: MDB	REV #: 0	DATE: MARCH, 2005
SCALE: 1:1000	JOB NO. LE0114		DRAWING NO. FIGURE 1

APPENDIX B

Soil Test Results (5)
City of Lethbridge Aggregate Specifications

ParklandGEO

SIEVE PARTICLE-SIZE ANALYSIS

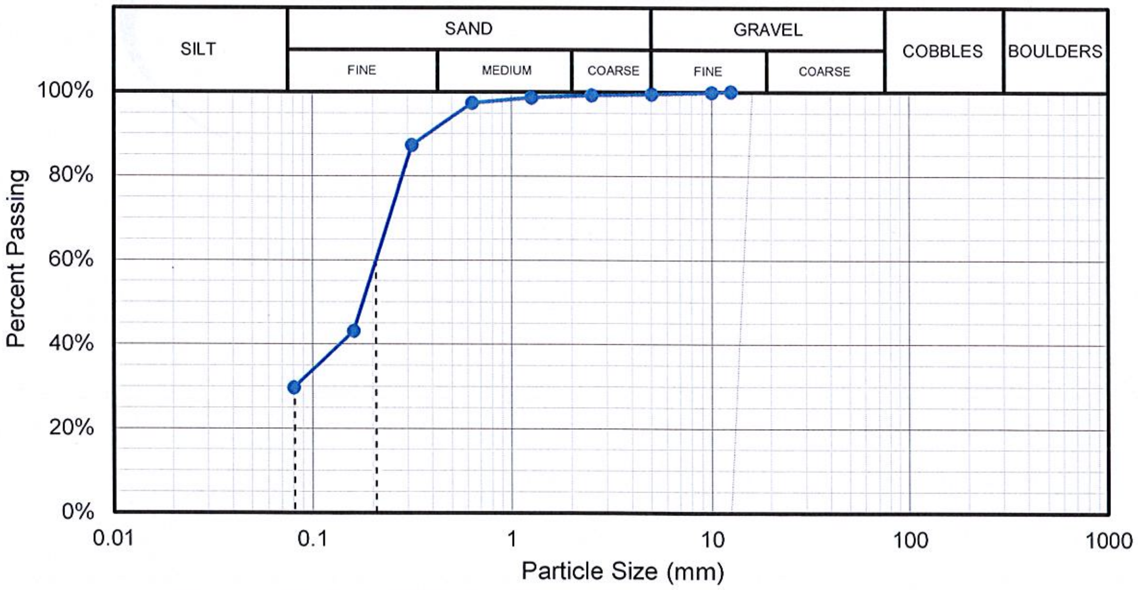
ASTM C136

PROJECT: Coaldale RCMP
PROJECT#: LE0114
CLIENT: RCMP
SOIL DESCRIPTION: Silty Sand

SAMPLED: June 20, 2017
TESTED: June 22, 2017
SAMPLE ID: 5D6
DEPTH: 12

SAMPLED BY: TB
LOCATION: BH5
DESIGNATION:
CLASS:

MASS MEASUREMENTS AND PERCENT PASSING	Sieve Size (mm)	Mass Retained on Sieve (g)	Cumulated Mass Retained (g)	Total Mass Finer (g)	Percent Passing	Specification Min	Specification Max
	80.0						
	63.0						
	50.0						
	40.0						
	25.0						
	20.0						
	16.0						
	12.5	0.0	0.0	1301.7	100.0%		
	10.0	3.1	3.1	1298.6	99.8%		
	5.0	4.1	7.2	1294.5	99.4%		
	2.5	3.8	11.0	1290.7	99.2%		
	1.25	7.4	18.4	1283.3	98.6%		
	0.630	16.9	35.3	1266.4	97.3%		
	0.315	130.1	165.4	1136.3	87.3%		
	0.160	575.2	740.6	561.1	43.1%		
	0.080	174.9	915.5	386.2	29.7%		
	Pan	17.1	932.6	369.1	0.0%	---	



RESULTS		
	Gravel	0.6%
	Sand	69.8%
	Silt & Clay	29.7%

GRAIN SIZE		
	D_{10}	
	D_{30}	0.08 mm
	D_{60}	0.21 mm

COEFF.		
	Uniformity, C_U	
	Curvature, C_C	

TECH:LV
CHECKED: TB

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ParklandGEO

LIQUID LIMIT, PLASTIC LIMIT, AND PLASTICITY INDEX

ASTM D4318 - Method A: Multi-Point

PROJECT: Coaldale RCMP Building

PROJECT#: LE0114

CLIENT: RCMP

SOIL DESCRIPTION:

SAMPLE DATE: 19-Jun-17

TEST DATE: 25-Jun-17

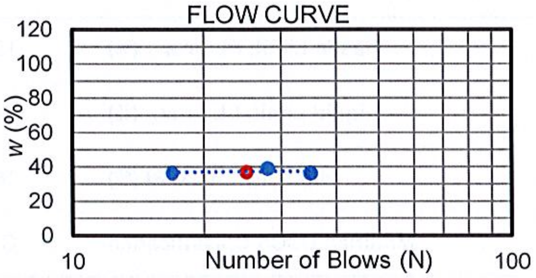
SAMPLE ID: 1G1

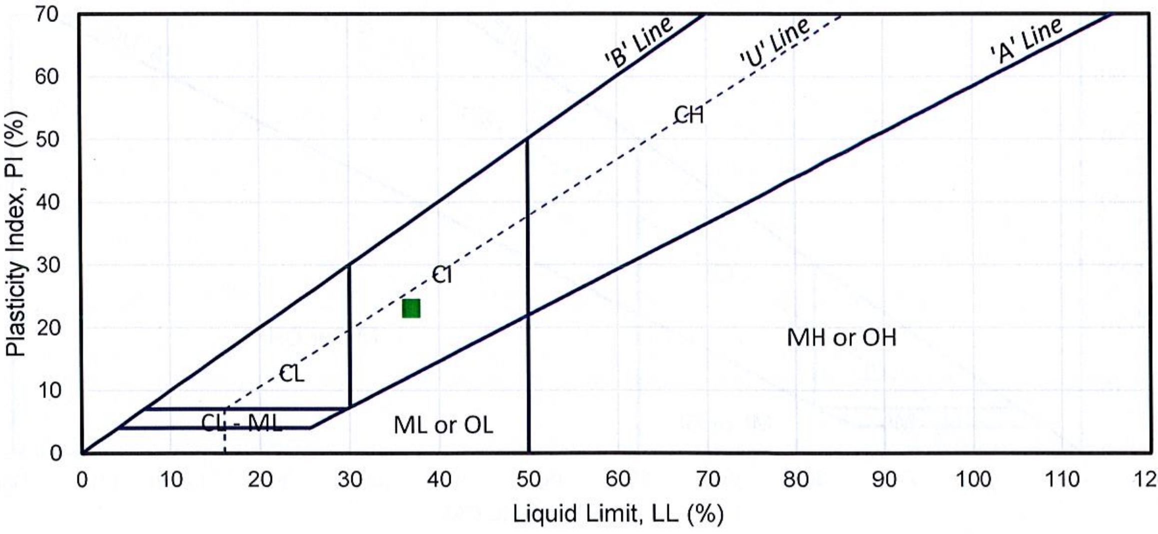
DEPTH: 1 m

PROCEDURE USED: Wet Preparation - Method A: Multi-Point

	AS RECEIVED	PLASTIC LIMIT				LIQUID LIMIT			
		1	2	3	4	1	2	3	4
Number of blows, N						35	28	17	
Container Number		F	B	K		C	H	T	
Tare Container, M_C (g)		30.652	30.599	30.678		30.759	30.866	30.75	
Wet Sample + Tare, M_{CMS} (g)		34.928	34.395	36.774		44.121	49.085	44.36	
Dry Sample + Tare, M_{CDS} (g)		34.403	33.942	36.059		40.580	43.995	40.75	
Dry Sample, M_S (g)		3.751	3.343	5.381		9.821	13.129	10.006	
Water, M_W (g)		0.525	0.453	0.715		3.541	5.090	3.603	
Moisture Content, w (%)		14.0	13.6	13.3		36.1	38.8	36.0	

Plastic Limit, PL or w_P (%)	14
Liquid Limit, LL or w_L (%)	37
Plasticity Index, PI (%)	23
Modified USCS Classification	CI





TECH:MV
CHECKED:TB

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ParklandGEO

LIQUID LIMIT, PLASTIC LIMIT, AND PLASTICITY INDEX

ASTM D4318 - Method A: Multi-Point

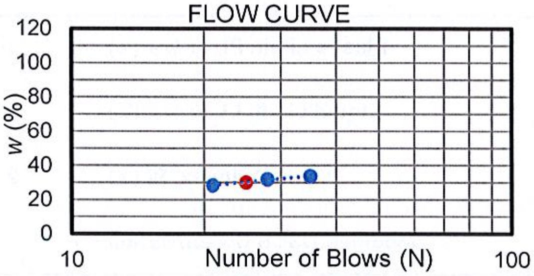
PROJECT: Coaldale RCMP Building
PROJECT#: LE0114
CLIENT: RCMP
SOIL DESCRIPTION:

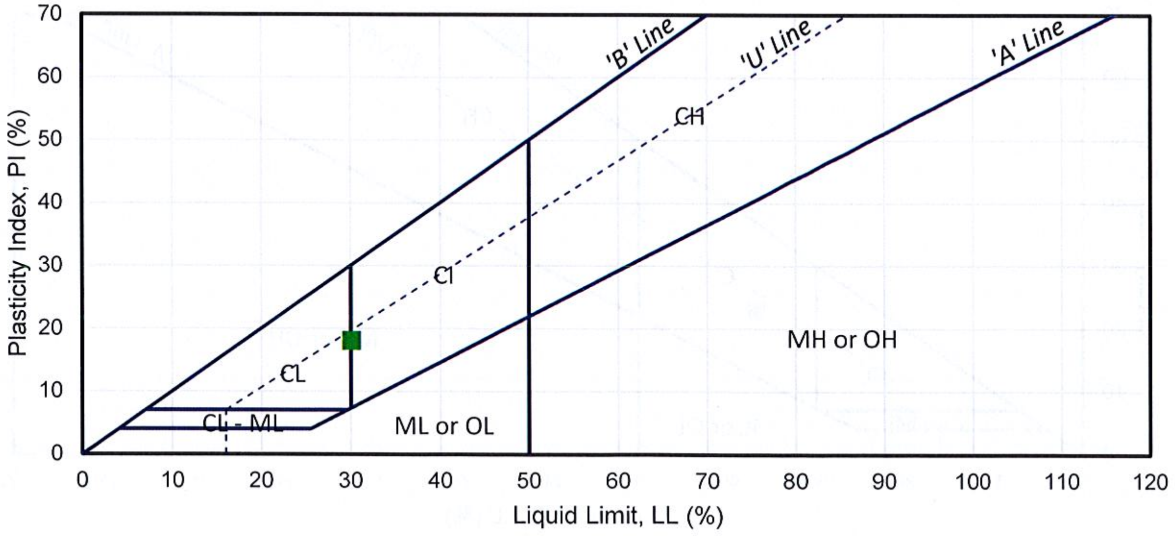
SAMPLE DATE: 19-Jun-17
TEST DATE: 25-Jun-17
SAMPLE ID: 10D5
DEPTH: 10.5

PROCEDURE USED: Wet Preparation - Method A: Multi-Point

	AS RECEIVED	PLASTIC LIMIT				LIQUID LIMIT			
		1	2	3	4	1	2	3	4
Number of blows, N						35	28	21	
Container Number		I	V	M		Q	E	N	
Tare Container, M_C (g)		30.805	30.838	30.829		30.737	30.774	30.68	
Wet Sample + Tare, M_{CMS} (g)		40.919	37.866	39.609		49.321	43.289	44.90	
Dry Sample + Tare, M_{CDS} (g)		39.816	37.088	38.681		44.640	40.275	41.78	
Dry Sample, M_S (g)		9.011	6.250	7.852		13.903	9.501	11.104	
Water, M_W (g)		1.103	0.778	0.928		4.681	3.014	3.112	
Moisture Content, w (%)		12.2	12.4	11.8		33.7	31.7	28.0	

Plastic Limit, PL or w_P (%)	12
Liquid Limit, LL or w_L (%)	30
Plasticity Index, PI (%)	18
Modified USCS Classification	CI





TECH:MV
CHECKED:TB

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Parkland**GEO**

LIQUID LIMIT, PLASTIC LIMIT, AND PLASTICITY

ASTM D4318 - Method A: Multi-Point

PROJECT: RCMP Coaldale
PROJECT#: LE0114
CLIENT: RCMP
SOIL DESCRIPTION: clay, and silt, trace sand

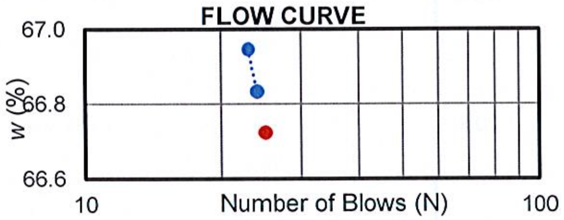
SAMPLE DATE: Jun 19/17
TEST DATE: Jun 27/17
SAMPLE ID: 2G1
DEPTH: 4.0m

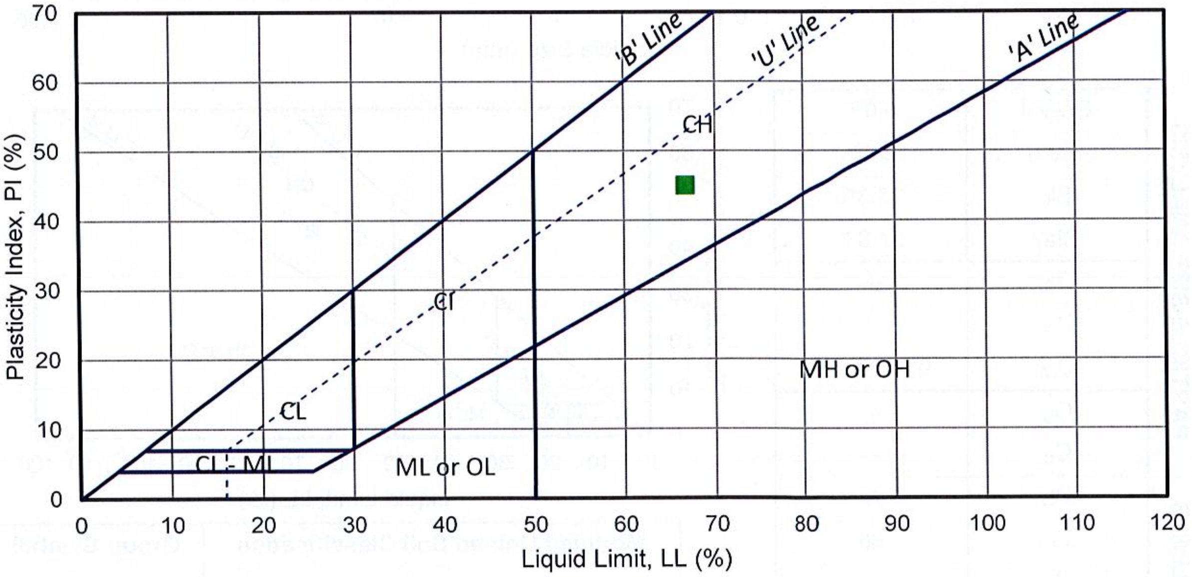
PROCEDURE USED: Dry Preparation - Method A: Mult-Point

	AS RECEIVED	PLASTIC LIMIT				LIQUID LIMIT			
		1	2	3	4	1	2	3	4
Number of blows, N						23	24		
Container Number		1	2	3		1	2		
Tare Container, M_C (g)		7.310	7.190	7.100		14.380	14.265		
Wet Sample + Tare, M_{CMS} (g)		9.715	9.750	9.680		25.315	25.935		
Dry Sample + Tare, M_{CDS} (g)		9.280	9.285	9.215		20.930	21.260		
Dry Sample, M_S (g)		1.970	2.095	2.115		6.550	6.995		
Water, M_W (g)		0.435	0.465	0.465		4.385	4.675		
Moisture Content, w (%)		22.1	22.2	22.0		66.9	66.8		

Plastic Limit, PL or w_P (%)	22
Liquid Limit, LL or w_L (%)	67
Plasticity Index, PI (%)	45
Modified USCS Classification	CH

FLOW CURVE





TECH: JB
CHECKED: ##

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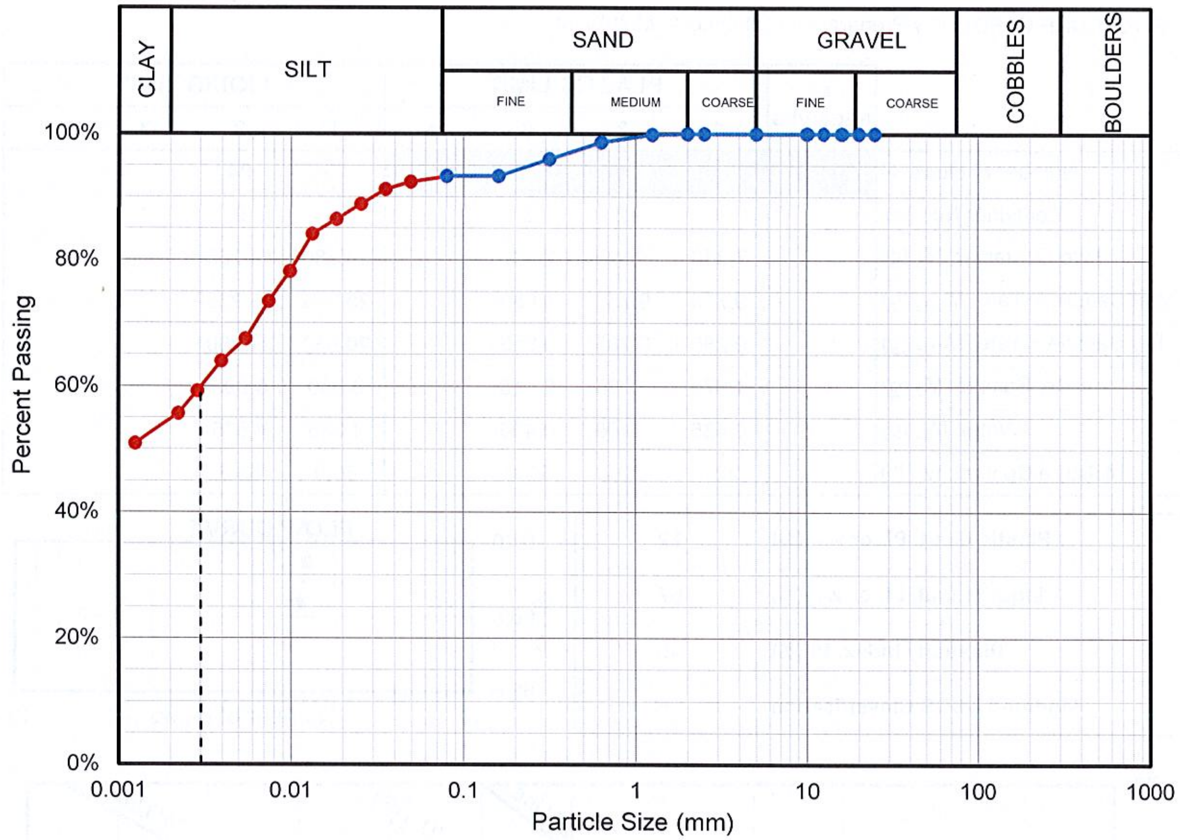
Parkland**GEO**

PARTICLE-SIZE ANALYSIS, LIQUID LIMIT, PLASTIC LIMIT, AND PLASTICITY

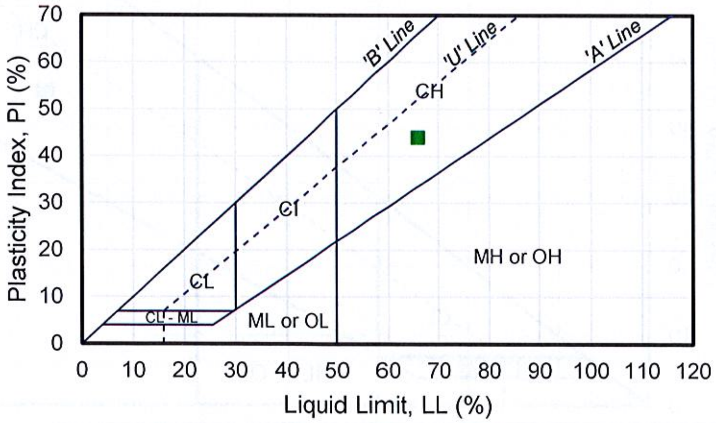
ASTM D422 & ASTM D4318

PROJECT: RCMP Coaldale
PROJECT#: LE0114
CLIENT: RCMP
SOIL DESCRIPTION: clay, and silt, trace sand

SAMPLE DATE: Jun 19/17
TEST DATE: Jun 27/17
SAMPLE ID: 2G1
DEPTH: 4.0m



PARTICLE-SIZE ANALYSIS	Gravel	0.0%
	Sand	6.8%
	Silt	38.3%
	Clay	54.9%
	D_{10}	---
	D_{30}	---
	D_{60}	0.0030 mm
	C_U	---
	C_C	---
LIMITS	PL	22
	LL	66
	PI	44



Modified Unified Soil Classification	Group Symbol
Fat clay	CH

TECH: JB
CHECKED: ##

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Down To Earth Labs Inc.
The Science of Higher Yields

Parkland GEO

Report # : 37374
Report Date: 6/27/2017
Received: 6/26/2017
Completed: 6/27/2017
Test Package: SFSO4

Project: Trevor Benson
PO:
Grower:
Field:

3510 6th Ave North
Lethbridge, AB T1H 5C3
403-328-1133
www.downtoearthlabs.com
info@downtoearthlabs.com

		Cust. Sample ID:	BH7-2	BH1-2	BH7-1
Analyte	**Units**	**Detection Limit**	170626J047	170626J048	170626J049
Sulfates	%	0.00001	1.271	1.822	0.098
Sulfates	mg/L	0.00001	12700	18200	981

Raygan Boyce - Chemist

Note: The analytical results pertain only to the submitted sample and may not be construed as an endorsement of the sampling method employed.
Methods are available upon request
BDL = Below Detection Limit

ParklandGEO

CITY OF LETHBRIDGE
AGGREGATE GRADATION SPECIFICATIONS

AGGREGATE DESCRIPTION

Asphalt Concrete Pavement Aggregate
Section 05140

- 20 mm ACP Aggregate (Mix Type I)
- 25 mm ACP Aggregate (Mix Type II)
- 16 mm ACP Aggregate (Mix Type III)

Granular Base Course (GBC)
Section 05020

- 25 mm GBC Aggregate

Granular Sub-Base
Section 05020

- 75 mm Select Aggregate

Screened GranularSub-Base
Section 05020

- 75 mm Select Aggregate

* Refer to Construction Specifications, City of Lethbridge

Revised January 2015

Aggregate Type		Asphalt Concrete Pavement			Granular Base Course	Granular Sub-Base	Screened Granular Sub-Base
		Mix I	Mix II	Mix III	25mm	75mm	75mm
SIEVE SIZE: Percent Passing Metric Sieve (CGSB 8-GP-2MM)	200000						
	150000						
	75000					100	100
	50000						
	40000						
	25000		100		100	65-100	60-100
	20000	100	85-95				
	16000	97-100	77-88	100	73-94		
	12500	85-95	65-80	90-100			
	10000	70-85	57-72	75-90	56-80	40-100	40-80
	5000	50-65	40-55	60-75	40-66	30-90	25-65
	4750						
	2500	40-50	30-42	60-75		15-35	
	1250	30-40	23-33	45-60	24-45		
	630	20-30	17-27	30-45		15-35	10-35
	315	15-23	12-22	22-36	13-27		
	160	6-16	6-15	15-27	9-19	5-15	5-15
	80	4-8	4-8	4-10	4-10	3-10	3-10
% Fracture (2 Face) Min.		80	60	80	60	N/A	N/A
LA Abrasion Max. Loss %		32	32	32	45	50	50
Liquid Limit Max. %		N/A	N/A	N/A	25	25	25
Plasticity Index Max. %		N/A	N/A	N/A	6	6	6
Lightweight Particles Max. %		1.5	1.5	1.5	5	5	5
CBR Min. %		N/A	N/A	N/A	80	20	20

LIMITATIONS

General Terms and Conditions

ParklandGEO

THE PARKLANDGEO CONSULTING GROUP GENERAL TERMS, CONDITIONS AND LIMITATIONS

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

1. STANDARD OF CARE - In the performance of professional services, ParklandGEO used the 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 in any manner.

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 or environmental consulting practice in this area. ParklandGEO will not be responsible for the interpretation by others of the information developed.

3. SITE INFORMATION - The CLIENT has agreed to provide all information with respect to the past, present and proposed conditions and use of the Site, whether specifically requested or not. The CLIENT acknowledged that in order for ParklandGEO to properly advise and assist the CLIENT, ParklandGEO has relied on full disclosure by the CLIENT of all matters pertinent to the Site investigation.

4. 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 has agreed that "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. ParklandGEO 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 has agreed that in the event that any such report is released to a third party, the above 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.

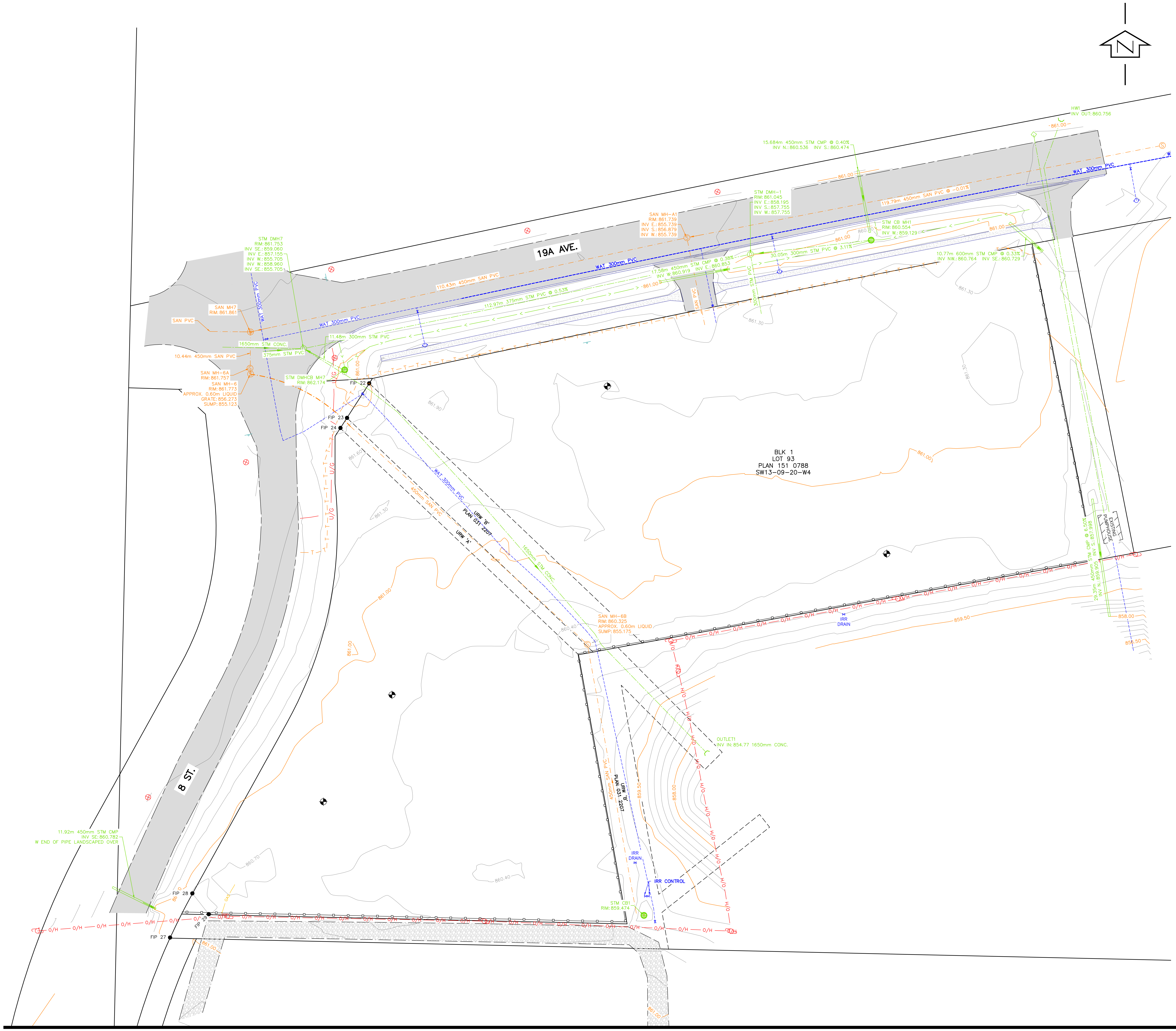
5. LIMITATIONS ON SCOPE OF INVESTIGATION AND WARRANTY DISCLAIMER
   There is no warranty, expressed or implied, by ParklandGEO that:
   a) the investigation uncovered all potential geo-hazards, contaminants or environmental liabilities on the Site; or
   b) the Site is entirely free of all geo-hazards or 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 geo-hazards or contaminants on the Site.

   The CLIENT acknowledged 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 laboratory testing program and 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 the 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 and in some cases the discovery of hazardous conditions and materials will require that certain regulatory bodies be informed. 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.

6. COST ESTIMATES - Estimates of remediation or construction costs can only be based on the specific information generated and the technical limitations of the investigation authorized by the CLIENT. Accordingly, estimated costs for construction or remediation are based on the known site conditions, which can vary as new information is discovered during construction. As some construction activities are an iterative exercise, ParklandGEO shall therefore not be liable for the accuracy of any estimates of remediation or construction costs provided.

7. LIMITATION OF LIABILITY - The CLIENT has agreed that to the fullest extent permitted by the law ParklandGEO's total liability to CLIENT for any and all injuries, claims, losses, expenses or damages whatsoever arising out of or in anyway relating to the Project is contractually limited, as outlined in ParklandGEO's standard Consulting Services Agreement. Further, the CLIENT has agreed that to the fullest extent permitted by law ParklandGEO is not liable to the CLIENT for any special, indirect or consequential damages whatsoever, regardless of cause.

8. INDEMNIFICATION - To the fullest extent permitted by law, the CLIENT has agreed to defend, indemnify and hold ParklandGEO, its directors, officers, employees, agents and subcontractors, harmless from and against any and all claims, defence costs, including legal fees on a full indemnity basis, damages, and other liabilities arising out of or in any way related to ParklandGEO's work, reports or recommendations.



LEGEND

CURB & GUTTER	═
SIDEWALK	═
EDGE OF PAVEMENT	— - — - —
EDGE OF GRAVEL	—·—·—·—
ASPHALT SURFACE	▨
GRAVEL SURFACE	▨
CONCRETE SURFACE	▨
O/H 3ø POWER & POLES	— O/H — O/H — O/H —
U/G POWER, LIGHT STANDARD	— U/G —⊗
GAS LINE	— GAS — GAS — GAS —
TELEPHONE LINE	— T — T — T — T — T — T — T —
TRANSFORMER, TELE. BOX, PEDESTAL	⊠
SIGN	⌒
SANITARY SEWER MAINLINE & MANHOLE	Ⓢ— · — · —
WATERLINE, HYDRANT, VALVE, CURBSTOP	⌽- - - - ⋈ - - - ⊗ - - -
IRRIGATION DRAIN & CONTROL VALVE	⋈ IRR DRAIN ⋈ IRR CONTROL
STORM SEWER MAINLINE, MANHOLE, CATCHBASIN, CULVERT, OUTFALL	Ⓓ— · — · — ▭ — ⊜
GRASSED SWALE	— < — < — < —
PROPERTY LINE	——
UTILITY EASEMENT	— — — — —
FOUND IRON PINS	● FIP 22
CONTOURS (0.3m INTERVAL)	— 861.00 —
BUILDINGS	▨
FENCE – CHAIN LINK	—□—□—□—
PIEZOMETER	⊕

NOTES:
1. DEEP UTILITY SIZES AND LOCATIONS PROVIDED BY TOWN OF COALDALE.
2. EXISTENCE AND LOCATION OF UNDERGROUND UTILITIES IS NOT GUARANTEED. IT IS THE CONTRACTOR'S RESPONSIBILITY TO DETERMINE THE EXISTENCE AND LOCATION OF ALL UTILITIES PRIOR TO GROUND DISTURBANCE.
3. CADASTRAL IS 150mm DIFFERENT THAN GPS MEASUREMENTS.
4. UTM 12 NORTH, NAD83, ELEVATIONS GEODETIC, GEOID MODEL GSD95, COORDINATES GRID.

No.	REVISION	DATE	REV. BY	APP. BY

ENGINEER

PERMIT

CICON ENGINEERING

DESIGN BY I.D.C.
DRAWN BY C.C.D.
CHECKED BY I.D.C.
APPROVED BY I.D.C.
SCALE 1:500

CLIENT/PROJECT
ROYAL CANADIAN MOUNTED POLICE
NEW RCMP DETACHMENT BUILDING – COALDALE, AB

TITLE
TOPOGRAPHIC MAP WITH SURFACE
AND UTILITY FEATURES

DATE 05/30/17
PROJECT No 505508
ISSUE No 1
SHEET 1 **OF** 1