DESKTOP GEOTECHNICAL ASSESSMENT
SOLAR FACILITY SITE
NAUJAAT, NU

FINAL REPORT

PREPARED FOR:
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PROJECT NUMBER:
NAU-G2201

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1.0 Introduction

Adaptive Baseline Geotechnical Ltd. (ABG) has carried out the following desktop geotechnical assessment of a solar facility site in Naujaat, NU. The desktop geotechnical assessment presented herein has been carried out in general accordance with the most recent editions of the National Standard of Canada CAN/BNQ 2501-500/2017 *Geotechnical Site Investigations for Building Foundations in Permafrost* and Canada Standards Association (CSA) PLUS 4011:19 *TECHNICAL GUIDE Infrastructure in permafrost: A guide for climate change adaptation*.

All third-party information reviewed as part of this assessment has been taken at face value. ABG's geotechnical personnel have not carried out a site-specific geotechnical borehole investigation or site visit as part of this assessment.

It is noted that available information throughout a community, as well as the design standards associated with climate change and foundation design in the north are constantly evolving and changing. For this reason, the findings and recommendations presented herein should only be considered relevant and applicable for a maximum of two years. After two years, ABG should be provided an opportunity to review and update this report prior to the findings or recommendations being used in support of design or construction at the site.

2.0 Project Background and Understanding

It is understood that a new utility scale solar facility will be constructed in the community. Solar arrays will be south facing, fixed tilt, bifacial modules including four inverters and a 1 MWh battery energy storage system. The purpose of the geotechnical assessment presented herein is to estimate subsurface conditions throughout the area of interest and provide geotechnical recommendations to support design and construction of foundations, associated roadway/parking areas and general site grading.

3.0 Scope of Services

ABG's scope of services for this desktop geotechnical assessment includes the following:

- ***Compilation and Review of Available Information:*** Compile all available information related to climate, site topography, surface drainage and subsurface conditions. Carry out a thorough review of the information pertinent to foundation design and construction at the site.
- ***Desktop Geotechnical Report:*** Prepare a desktop geotechnical assessment report detailing the anticipated site conditions and geotechnical recommendations to support design and construction of the chosen foundation type, associated roadway/parking areas and general site grading.

4.0 Available Information

ABG has reviewed the following available information as part of this desktop geotechnical assessment:

1. (Canadrill, 2021a). Geotechnical Investigation Rev. 1, New Sewage Lagoon, Naujaat, NU;
2. (Canadrill, 2021b). Pile Installation Summary Tables, New 5-Plexes, Naujaat, NU;

3. (Canadrill, 2020). Pile Installation Summary Report, 3 New 5-Plex Units, Naujaat, NU;
4. (Canadrill, 2018a). Pile Installation Summary Report, Bell Cellular Tower, Naujaat, NU;
5. (Canadrill, 2018b). Pile Installation Summary Report, New 5-Plex Unit, Lots 8 & 9, Naujaat, NU;
6. (Canadrill, 2018c). Pile Installation Summary Report, Satellite Antenna, Naujaat, NU;
7. (exp, 2016). Summary of Pile Foundation Installations, Water Pump House, Lot 257, Naujaat, NU;
8. (exp, 2014). Summary of Pile Foundation Installations, Two New 10-Plex Buildings, Naujaat, NU;
9. Aerial photography, National Air Photo Library (1969, 1976, 1984, and 1995);
10. Available satellite imagery, Google Earth (2017, 2021);
11. Available LiDAR survey for the community;
12. Environment Canada, historical weather data for the community of Naujaat, NU; and
13. Studies and literature related to the distribution of saline permafrost and ground temperature data throughout Nunavut (i.e. Canadian Geotechnical Journals).

5.0 Historical Climate and Permafrost Conditions

Naujaat is located at approximately 66° 31’ N and 86° 14’ W on the shore of Hudson Bay, in the Kivalliq Region of Nunavut. Based on current permafrost mapping, the community is located within the zone of continuous permafrost.

Mean Annual Air Temperature (MAAT) and Indices: A review of Environment Canada climate records for the community revealed a relatively complete set of historical monthly air temperatures spanning the period from 1981 to 2010. The data indicates the MAAT over this time-period was -12.0°C and the average thawing and freezing indices were about 600°C-days and 4992°C-days respectively.

Active Layer Thickness: Based on the above-noted historical air temperature data, simplified empirical methods and active layer thickness measurements, it is estimated that the active layer currently varies between approximately 1.3 and 2.1 m, depending on site-specific variables (such as surficial cover, site drainage, sun exposure and in-situ moisture content). Based on nearby information, we have assumed an active layer thickness of 1.8 m at the site.

Mean Annual Ground Temperature (MAGT): Based on a review of available ground temperature data throughout the community, we have assumed a current MAGT of -8.5°C at the site.

6.0 Anticipated Site and Soil Conditions

The proposed solar facility site is to be constructed on undeveloped land located on the eastern edge of the community, approximately 900 m from the coastline. Based on the survey data picked up by ABG in July 2022, the site elevation ranges from 25 to 35 meter above sea level (masl). The site lies in a shallow valley between two bedrock outcrops to the southwest (32 masl) and the northeast (35 masl). From the middle of the site, the valley slopes towards a small pond in northwest of the site from an elevation of 29.5 masl to an elevation of 27 masl. The pond then drains through a culvert passing under the road adjacent to the northern side of the site. The site also slopes from the middle of the site towards a wetland to the southeast from an elevation of 29.5 masl to an elevation of 25 masl. The site location is shown on Figure 1.

A summary of anticipated surface and subsurface conditions is included below. It is recommended that the actual subsurface conditions be further evaluated/confirmed at the time of construction by a geotechnical engineer experienced with northern construction and registered with Northwest Territories and Nunavut Association of Professional Engineers and Geoscientists (NAPEG), or their representative.

6.1 Anticipated Overburden

Based on our review of available information, ABG anticipates that the overburden throughout the site will consist of either bedrock outcrop or original rootmat/topsoil underlain by ice-rich native soils (likely silty or clayey sand) over shallow bedrock, with some deeper pockets of potentially soft saturated clayey sand along the lowest lying areas of the site, mid-way between the exposed bedrock ridges. Overburden moisture contents are anticipated to range from 5 to 20%, with the higher end of that range being associated with ice-rich permafrost soils at depth. Overburden thickness at the site is unknown; however, it is assumed to be less than 5 m based on our experience.

6.2 Anticipated Bedrock Geology

Based on available geological maps for the area, it is anticipated that bedrock throughout the community would consist of granite and metamorphic bedrock. Based on a review of available satellite imagery, air photos and site survey, the site is bordered by bedrock outcrops to the northeast and southwest. Based on a review of available information, exposed and shallow bedrock has been encountered throughout the area of interest with the upper portion of the bedrock surface often very severely fractured and weathered.

6.3 Anticipated Groundwater

Based on available information and general topography of the area, groundwater flow through the site is anticipated to be light to moderate during the spring freshet and may even be ponded below grade, travelling along the bottom of the active layer or top of sound bedrock during thaw. It is anticipated that piling may have to contend with higher than average amounts of groundwater at the site during thaw. It is also anticipated that groundwater levels will fluctuate with seasonal weather trends, during precipitation events, and with significant site disturbance and construction activities.

6.4 Anticipated Porewater Salinity

Based on our review of available salinity information, we have assumed an average porewater salinity value of 10 parts per thousand (ppt) at the site.

7.0 Climate Change in Foundation Design

ABG anticipates that the current maximum active layer thickness throughout the site is approximately 1.8 m and the current MAGT is -8.5°C. Changes to the active layer thickness and MAGT throughout the life of the structure will depend on many variables, possibly including but not limited to actual current values, changes to MAAT, snow cover, precipitation, surface/groundwater flow, material gradation and in-situ ice content.

CSA PLUS 4011:19 provides that under a high green house gas scenario the MAAT in Naujaat is estimated to increase by approximately 2.5°C over the next 30 years (by 2052) compared to the historic temperature trends which were available up to 2010. It is noted however, that recent research infers that greenhouse gas emissions over the next 30 years and beyond may be even higher than previously anticipated and new scenarios continue to be produced by global experts. Therefore, accurately estimating what the active layer thickness and MAGT will be 30 years from now is well beyond the scope of this assessment.

To support the current project, we have adjusted the historical temperature data to incorporate the above-noted changes to the MAAT and utilized the same simplified empirical methods from Section 5.0 to generate an estimated maximum active layer thickness 30 years from now. We have also assumed (conservatively) that the MAGT will change in step with the MAAT over this period. The process results in future estimated values for the maximum active layer thickness and MAGT of 2.3 m and -6.0°C, respectively. Given the inherent uncertainties surrounding the effects that climate change and site development will have on active layer thickness and MAGT at the site, we recommend introducing some additional conservatism by the way of engineering judgement. For this reason, we have used a design active layer thickness of 2.8 m and a design MAGT of -5.5°C for pile design at the site.

It is further recommended that a series of thermistors be installed along with select pile installations such that ground temperature monitoring can be carried out to establish the actual site-specific ground temperature profile over time. In this way, the assumptions made to support design can be confirmed based on the real-world conditions during the pile installations and any issues that may occur can be better understood and dealt with accordingly.

It is noted that CSA PLUS 4011:19 states *"The requirement for monitoring, reporting, and reacting to any changes that are noted must be recognized early in the project. The responsibilities need to be defined at the project outset and budgets allocated to collect and summarize the data. An annual review by the geotechnical engineer is recommended with more frequent reviews if undesirable trends appear. Monitoring is pointless unless the data collected are evaluated".* This speaks to the importance of implementing a proper and consistent ground temperature monitoring program that includes review and input from qualified geotechnical personnel as part of responsibly addressing climate change in relation to foundation design and maintenance.

8.0 Discussion and Recommendations

Based on our review of available information as discussed herein, the site is suitable for the use of steel pipe pile foundations. The type of pipe pile installation (rock socket or adfreeze) depends on the actual depth to bedrock beneath the site. Based on the available information, it is anticipated that bedrock will be encountered at surface or just below surface; therefore, we recommend that the solar facility be founded atop rock socket piles. Recommendations to support the design and construction of rock socket steel pipe piles, as well as driveway/parking areas and general grading at the site are included below.

8.1 Rock Socket Steel Pipe Piles

Rock socket steel pipe piles carry the applied loads through a grouted bond between the pipe pile and the bedrock socket. The rock socket bond is used to resist both compression and uplift loads. The following table provides unfactored Ultimate Limit States (ULS) and Serviceability Limit States (SLS) rock socket bond capacities for anticipated pile diameters, as well as the required minimum embedment from a geotechnical perspective.

TABLE 1 - Rock Socket Pile Design Parameters

Pile Outside Diameter (mm)	Unfactored ULS Grout-to-Rock Bond Capacity (kPa)	SLS Grout-to-Rock Bond Capacity (kPa)	Minimum Embedment into Sound Bedrock (m)
114 to 141	1500	N/A	1.0

Notes: 1) ULS unfactored bond capacity is based on the use of 30 MPa grout and slots through the rock socket.
2) ULS geotechnical resistance factor of 0.4 should be applied for compression loads.
3) ULS geotechnical resistance factor of 0.3 should be applied for tension loads.
4) Minimum embedment is based on engineering judgement taking into consideration the requirement to resist a design frost jacking force of 150 kPa through the design active layer thickness and incorporating some additional embedment to help counter the possible unknown effects of climate change.

The minimum embedment presented above is intended to provide enough resistance to avoid frost jacking of piles based on the design active layer thickness, as well as some additional embedment incorporated based on our engineering judgement to further help counter the possible unknown effects of climate change over time as much as practical.

Given the difficulties in accurately estimating the active layer thickness, it is also recommended that a suitable bond breaker be provided through the design active layer thickness of each pile, or to within 0.3 m of the top of the rock socket bond zone, as a secondary measure. The bond breaker can be excluded if sound bedrock is encountered within 1 m of design grade.

It is noted that the frost jacking force of 150 kPa is a recommended design force based on anticipated soil conditions and the use of steel pipe piles; therefore, the design frost jacking force does not require the application of a load factor during pile design and shall be considered together with other uplift forces acting on the structure (i.e. wind) to determine the actual pile embedment/socket length required for final design.

For lateral design, we have estimated values of horizontal subgrade modulus based on the anticipated stratigraphy (where there is not exposed bedrock) and our installation methods, as shown in the following table. If additional lateral capacity is required, our office can work with the structural designer to determine a more specific lateral load capacity of the piles given the anticipated design loads and potentially less conservative assumptions (e.g. L-Pile analyses).

TABLE 2 - Estimated Horizontal Subgrade Modulus Values

Assumed Soil Profile	Depth (mbg)	Coefficient of Subgrade Reaction (k_s, kN/m^3)
Sand (thawed, active layer, above groundwater)	0.0	0
Sand (thawed, active layer, above groundwater)	0.1	1,000

TABLE 2 - Estimated Horizontal Subgrade Modulus Values

Assumed Soil Profile	Depth (mbg)	Coefficient of Subgrade Reaction (k_s, kN/m^3)
Sand (thawed, active layer, above groundwater)	0.5	7,000
Sand (thawed, active layer, above groundwater)	1.0	9,000
Sand (thawed, active layer, below groundwater)	1.5	25,000
Sand (thawed, active layer, below groundwater)	2.0	48,000
Sand (thawed, active layer, below groundwater)	2.5	78,000
Sand (thawed, active layer, below groundwater)	2.8	90,000
Sand (frozen, permafrost)	2.9	92,000
Sand (frozen, permafrost)	3.5	95,000
Sand (frozen, permafrost)	4.0	130,000
Sand (frozen, permafrost)	4.5	135,000
Sand (frozen, permafrost)	5.0	140,000
Bedrock (grouted socket)	5.1	904,000
Bedrock (grouted socket)	5.5	975,000
Bedrock (grouted socket)	6.0	1,063,000

Notes: 1) Coefficient of subgrade reaction calculated for a 141 mm steel pipe pile.

It is noted that the bearing capacity of the piles may be reduced by improper seating of the pile on the bedrock or by poor bond at the grout/pile interface; therefore, the portion of the pile to be grouted must be free of paint, lacquer, oil and dirt. Each pile shall be vibrated into place to assure a good seat is achieved on sound bedrock.

To avoid group effect considerations, individual piles should be at least 3 pile diameters apart center to center. Further recommendations pertaining to consideration of group effect can be provided upon request if closer pile spacing is required.

The piles should be installed in pre-drilled oversize holes at least 50 mm larger than the pile diameter. The bedrock socket should be filled with an approved fast setting arctic grout, such as SIKA ARCTIC 100 or equivalent. The procedure for mixing, handling and installing the grout should be in accordance with the manufacturer's recommendations. Mix ratios of the grout should be closely monitored throughout placement to ensure proper ratios are utilized. Test cubes of the prepared grout should be cast and tested for Unconfined Compressive Strength (UCS) each day. If grout installation to the bottom of the rock socket is carried out in the wet, the grout must be tremied to the bottom to avoid dispersion and dilution.

The piles should be installed open ended through the grout and vibrated to assure a good seat as noted above. The remaining annular space between the pile and the hole should be filled with sand to the final grade. Drill cuttings are suitable for this purpose. Loads should not be applied to the pile for at least three days after installation.

For a building supported on rock socket piles, a multi-bead thermistor should be installed to a depth below the assumed design active layer in one rock socket pile with adequately spaced thermistor beads. This thermistor string will allow for the verification of active layer design assumptions in addition to long-term monitoring, as recommended in Section 7.0.

The base of the structure should be at least 200 mm above final grade to provide an adequate gap between the structure and the ground surface to avoid unexpected uplift forces during seasonal heave of near-surface soils. If an adequate gap cannot be provided, the use of a suitable thickness of void form between the underside of the structure and the ground surface can be considered. It is noted that when heated structures are built on rock socket steel pipe piles or founded directly on grade, some settlement of grades around the building would be expected over time.

8.2 Parking Area, Site Grading and Drainage

The final site grades should be designed to eliminate the potential for ponding water around and ensure positive drainage away from the foundation elements onsite. In general, it is recommended that the final grades provide at least 3% grade away from the foundation elements in all directions.

It is anticipated that all driveway/parking areas will be gravel surfaced and the near surface soils onsite will consist predominantly of silty sand. Therefore, fill pads for the parking area and general site grading purposes should perform well and require minimal regrading each year if constructed in accordance with the recommendations included below.

It is recommended that all driveway/parking areas receive at least 200 mm of Type 1 material (surface/base course), underlain by at least 300 mm of Type 2 material (sub-base), placed and compacted atop an approved subgrade. Any heavy-duty traffic areas should receive an additional 300 mm of compacted Type 2 material (600 mm Type 2 sub-base material total) unless the subgrade is bedrock. Areas of bedrock subgrade should receive at least 150 mm of compacted Type 2 sub-base material to reduce the potential of the bedrock surface profile being reflected throughout the surface of the roadway/parking areas.

The satisfactory performance of driveway/parking areas will be dependant on the provision of adequate surface/subsurface drainage via professionally designed ditches, swales and culverts at the site.

It is recommended that cuts into the native soils be avoided and design grades be achieved by building atop the native grades of the site. Prior to any material placement, the native subgrade should be inspected by qualified geotechnical personnel to determine if any areas of concern exist (i.e. areas of extensive soft soils) and if so, the extent and nature of the concern should be discussed with the design team to determine the best area-specific treatment. Priority should always be given to maintaining the native organic mat wherever possible, even if some short-term regrading may be required as this material settles under use.

If it is determined that excavation and replacement is warranted to improve the native subgrade, the excavation should conform to the requirements of the Occupational Health and Safety Act (OSHA) and consider the potential for underlying permafrost soils to thaw/soften during excavation creating an increasingly worse situation. Excavations should be limited to only those areas deemed necessary and carried out in stages such that excavation and replacement back to current grade or higher occurs within hours (dependant on the observed rate of thaw and native soil behaviour).

All driveway/parking area fill material should be placed in maximum 300 mm thick lifts and compacted to 98% of the Standard Proctor Maximum Dry Density (SPMDD) within 300 mm of the subgrade surface and 95% of the SPMDD below this depth. The Type 1/Type 2 granular materials should be compacted to 100% of the SPMDD. Density testing is recommended to confirm each lift receives an adequate level of compaction prior to subsequent lifts. All permanent slopes for driveway/parking areas prepared as outlined herein shall be 2.5 horizontal:1 vertical (2.5H:1V) or gentler, unless approved by the geotechnical engineer of record for the project.

The following table below provides recommended gradation requirements for subgrade materials at the site. Other gradations may also be suitable, subject to review and approval by the geotechnical engineer of record.

TABLE 3 - Recommended Gradation for Type 1, Type 2 and Select Subgrade Materials

Property	ASTM Test Method	Type 2 (Sub-Base)	Type 1 (Base)	Select Subgrade
Gradation (sieve/% passing)	–	–	–	–
150 mm	C136	–	–	100
75.0 mm	C136	100	–	–
37.5 mm	C136	–	–	–
25.0 mm	C136	50 – 100	100	50 – 100
19.0 mm	C136	–	75 – 100	45 – 100
9.5 mm	C136	–	50 – 85	–
4.75 mm	C136	20 – 55	35 – 65	20 – 70
2.0 mm	C136	–	25 – 50	–
0.425 mm	C136	5 – 35	15 – 30	5 – 45
0.300 mm	C136	–	–	–
0.150 mm	C136	–	–	–
0.075 mm	C117	0 – 8	5 – 8	0 – 20

9.0 Site Classification for Seismic Site Response

Based on the anticipated subsurface conditions, the site can be classified as “Class C” for seismic site response in accordance with the requirements of Section 4.1.8.4 of the National Building Code of Canada (NBCC), 2015. In addition, the overburden soils are also considered to be non-liquefiable.

10.0 Requirement for Qualified Geotechnical Monitoring

As noted above, inspection and monitoring of foundation installations is an important component of the design process. Specialty quality assurance services (field observations and testing) during construction are a necessary part of the evaluation of sub-surface conditions and site preparation activities. This is especially true considering that the geotechnical site assessment outlined herein is based on a desktop review of available information. Qualified geotechnical personnel must assess the site at some point during the design and construction process.

Section 4.2.2.3 of the NBCC states that *"a field review shall be carried out by the designer or by another suitably qualified person to ascertain that the subsurface conditions are consistent with the design and that construction is carried out in accordance with the design and good engineering practice. The review required by Sentence (1) shall be carried out on a continuous basis during the construction of all deep foundation units with all pertinent information recorded for each foundation unit."*

Therefore, each steel pipe pile installation requires full-time monitoring by qualified geotechnical personnel familiar with air-track drilling procedures and permafrost soils, to confirm subsurface conditions are consistent with the design and that construction is carried out in accordance with design. During rock socket pile installations, onsite geotechnical personnel shall monitor for bedrock depth/quality and observe grout mixing procedures closely, sampling the grout daily for unconfined compressive strength tests.

It is also recommended that qualified geotechnical personnel be onsite throughout parking/driveway area preparation to assure the native subgrade preparation and material placement/compaction meets project requirements and intended purpose.

It is noted that ABG is available to provide a suitably qualified field engineer onsite during pile installations to work under the direction and guidance of the undersigned upon request, to confirm subsurface conditions, install thermistors along select piles and document each installation in accordance with the above-noted NBCC requirements.

11.0 Closure

The use of this report is subject to the attached statement of general conditions. It is the responsibility of Kivalliq Alternative Energy Ltd. who is identified as “the Client” within the statement of general conditions and its agents to review the conditions and to notify Adaptive Baseline Geotechnical Limited should any of these not be satisfied. The statement of general conditions addresses the use of the report, basis of the report, standard of care, interpretation of site conditions, varying or unexpected site conditions, planning, design and construction.

We trust the information contained herein is adequate for your present purposes. Should you have any questions about the contents of the report, or if we can be of any further assistance, please do not hesitate to contact the undersigned at your convenience.

Sincerely,

Adaptive Baseline Geotechnical Ltd.

PERMIT TO PRACTICE
ADAPTIVE BASELINE GEOTECHNICAL LTD.
Signature
Date 05-JAN-2023
PERMIT NUMBER: P 1501
NT/NU Association of Professional Engineers and Geoscientists

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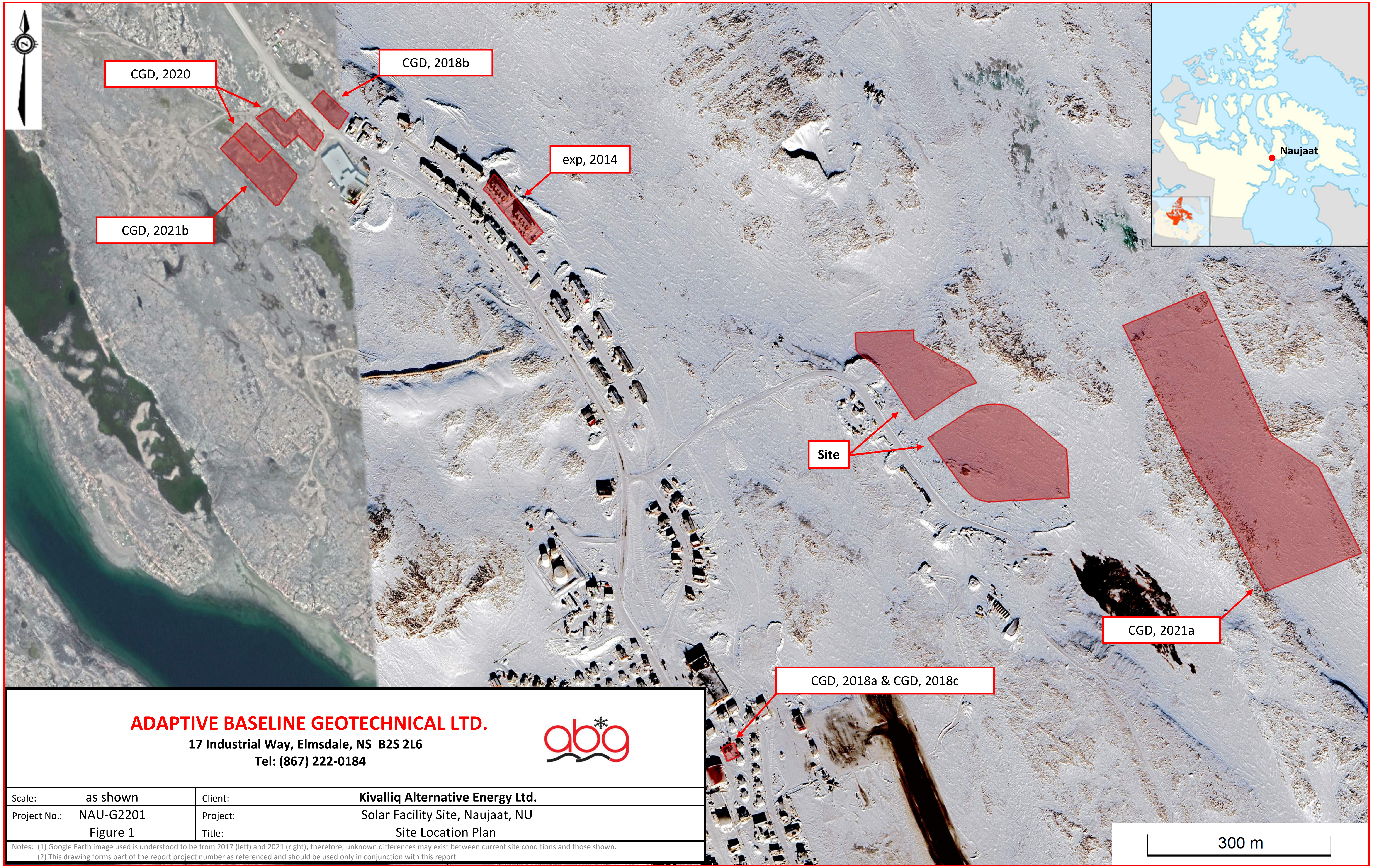
ATTACHMENTS



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Scale: as shown		Client:	**Kivalliq Alternative Energy Ltd.**
Project No.: NAU-G2201		Project:	Solar Facility Site, Naujaat, NU
Figure 1		Title:	Site Location Plan

Notes: (1) Google Earth image used is understood to be from 2017 (left) and 2021 (right); therefore, unknown differences may exist between current site conditions and those shown.
(2) This drawing forms part of the report project number as referenced and should be used only in conjunction with this report.

STATEMENT OF GENERAL CONDITIONS

Use of this Report: This report has been prepared for the sole benefit of the Client or its agent and may not be used by any third party without the express written consent of Adaptive Baseline Geotechnical Limited (ABG Ltd.) and the Client. Any use which a third party makes of this report is the responsibility of such third party.

Basis of the Report: The information, opinions, and/or recommendations made in this report are in accordance with ABG Ltd.'s present understanding of the site-specific project as described by the Client. The applicability of these is restricted to the site conditions encountered at the time of the investigation or study. If the proposed site-specific project differs or is modified from what is described in this report, or if the site conditions are altered, this report is no longer valid unless ABG Ltd. is requested by the Client to review and revise the report to reflect the differing or modified project specifics and/or the altered site conditions.

Standard of Care: Preparation of this report and all associated work was carried out in accordance with the normally accepted standard of care in the province or territory of execution for the specific professional service provided to the Client. No other warranty is made.

Interpretation of Site Conditions: Where ABG Ltd. has carried out a test pit or borehole field program, the soil, rock or other material descriptions and statements regarding their condition made in this report are based on site conditions encountered by ABG Ltd. at the time of the work and at the specified testing and/or sampling locations. Classifications and statements of condition have been made in accordance with normally accepted practices which are judgement in nature; no specific description should be considered exact, but rather reflective of the anticipated material behavior. Extrapolation of in-situ conditions can only be made to some limited extent beyond the sampling or test points. The extent depends on variability of the soil, rock and groundwater conditions as influenced by geological processes, construction activity and site use. In the case of a desktop assessment the previous reports and information prepared by other parties has been taken at face value.

Varying or Unexpected Conditions: Should any site or subsurface conditions be encountered that are different from those described in this report or encountered at the test locations, ABG Ltd. must be notified immediately to assess if the varying or unexpected conditions are substantial and if reassessments of the report conclusions or recommendations are required. ABG Ltd. will not be responsible to any party for damages incurred as a result of failing to notify us that differing site or sub-surface conditions are present upon becoming aware of such conditions.

Planning, Design or Construction: Development or design plans and specifications should be reviewed by ABG Ltd. sufficiently ahead of initiating the next project stage (property acquisition, tender, construction, etc.) to confirm that this report completely addresses the elaborated project specifics and that the contents of this report have been properly interpreted. Specialty quality assurance services (field observations and testing) during construction are a necessary part of the evaluation of sub-surface conditions and site preparation works. Site work relating to the recommendations included in this report should only be carried out in the presence of a qualified geotechnical engineer; ABG Ltd. cannot be responsible for site work carried out without being present.