FINAL GEOTECHNICAL INVESTIGATION

**CS-2: UPPER CANADA SOLAR 02**

5061 COUNTY ROAD 29

ELIZABETHTOWN-KITLEY, ONTARIO

| | |
|---|---|
| Date: **February 22, 2011** | Reference No**.: T020822-A2** |

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Reference No.: T020822-A2

February 22, 2011

Mr. Jeff Roy
Project Manager
Canadian Solar Solutions Inc.
67A Sparks Street, Suite 300
Ottawa, ON K1P 5A5

Re: Final Geotechnical Investigation
**CS-2: Upper Canada Solar 02**
5061 County Road 29
Elizabethtown-Kitley, Ontario

Dear Mr. Roy,

In accordance with your instructions, Inspec-Sol Inc. (**Inspec-Sol**) has completed a Final Geotechnical Investigation at the above-mentioned Site and is pleased to present the findings.

We trust this information meets with your approval. Please do not hesitate to contact us should any questions arise.

Yours very truly,
**INSPEC-SOL INC.**

Joseph B. Bennett, P. Eng.
Vice-President

SD/nc

Enclosures

Dist: Mr. Jeff Roy – Email - (jeff.roy@canadian-solar.com)

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## 1.0 INTRODUCTION

Inspec-Sol Inc**.** (**Inspec-Sol**) was authorized to carry out a Final Geotechnical Investigation for the proposed Upper Canada Solar 02 project, which is located at 5061 County Road 29 in the township of Elizabethtown-Kitley, Ontario (Site). Authorization to proceed with this study was provided by Mr. Jeff Roy representing Canadian Solar Solutions Inc. (Client).

The purpose of the investigation was to carry out an investigation, to evaluate the subsoil stratigraphy found at test locations; and based upon the collected data, provide recommendations concerning foundation options and associated bearing pressures, as well as provide comments to assist designers and the Client in regard to excavations, backfill, access roads, underground services, and construction field review.

This report has been prepared with the understanding that the design will be carried out in accordance with all applicable codes and standards. Any changes to the described project will require a review by **Inspec-Sol** to assess the impact of the changes on the report recommendations provided herein.

The scope of work for **Inspec-Sol** consisted of the following activities:

- **Desktop Study:** Consultation of publicly available soil and bedrock mapping in the vicinity of the Site, if available.
- **Field Work:** Excavation of nine (9) test pits, and advancement of eight (8) boreholes across the Site;
- **Laboratory Testing:** Submittal of five (5) grab samples to the laboratory for analysis of pH, redox potential, sulfate concentration, sulphide concentration, chloride concentration, and resistivity; and
- **Analysis and Report:** Review of field results, laboratory testing, and preparation of one (1) preliminary and one (1) final geotechnical report to provide comments and recommendations for design and construction of the proposed solar farm.

## 2.0 SITE DESCRIPTION

The proposed solar farm is located at a municipal address of 5061 County Road 29 in the township of Elizabethtown-Kitley, Ontario. The property is defined as part of Lot 25 Concession 5. Our understanding of the project boundaries is based on the "Aerial/Plan View PV System Layout" (Canadian Solar, Ref No. OICI-2010-0067, dated October 13, 2010), received from the Client. The location of the Site within the Elizabethtown-Kitley Township is shown on the attached *Site Location Map: T020822-A2-1*

The Site is currently undeveloped and has grass or small brush cover. The Site topography is relatively flat and is at the same general elevation of all adjoining properties.

## 3.0 GEOLOGICAL DISCUSSION

The desktop study component of the Preliminary Geotechnical Investigation consisted of consulting publicly available soils and bedrock maps, as well as previous **Inspec-Sol** reports. For this Site, the following resources were consulted:

- Ontario Well Registry Data, updates ongoing;
- "Bedrock Map – County of Leeds, ON" Ontario Department of Mines, 1922; and
- Two (2) nearby **Inspec-Sol** reports (Addison and Rows Corners).

Based on the above sources, the local soils were expected to consist of a thin layer of stiff silty clay or Glacial Till overburden overlying limestone bedrock. Bedrock was estimated at a depth of approximately 1.0m to 2.0m below the surface grade. The bedrock in this area was expected to consist of a limestone of the Pamelia Formation or a sandstone.

The groundwater level is reported to be ranging in depth from 5.0m to 10m below the ground surface.

## 4.0 FIELDWORK

### *4.1 Test Pit Program*

The test pit component of the fieldwork for the Preliminary Geotechnical Investigation was undertaken on October 29, 2010, by means of a rubber tracked mini excavator, under the supervision of **Inspec-Sol** field staff. A total of nine (9) test pits were dug down to the bedrock surface. They were dug to an average depth of 0.5m below the existing ground surface.

### *4.2 Borehole Program*

The borehole component of the fieldwork for this Geotechnical Investigation was undertaken on January 5 and 6, 2011, by means of a specialized track mounted drill rig adapted for soil sampling. Boreholes were advanced through the overburden using standard 50 mm split spoons and hollow stem augers. Standard penetration test (SPT) values were recorded on the borehole logs, where applicable. Boreholes were advanced into bedrock using double walled coring techniques. The drilling was completed under the supervision of **Inspec-Sol** field staff. Four (4) holes were advanced to practical auger refusal on the bedrock surface, and the remaining four (4) holes had rock core retrieved to a minimum depth of 3.0m into bedrock.

The locations of these test pits and boreholes can be found on the attached *Test Pit and Borehole Location Plan: T020822-A1-2.* A levelling survey of the test pit locations was not completed as part of this Geotechnical Investigation.

## 5.0 LABORATORY TESTING

### *5.1 Chemical Testing*

The chemical testing component of this Geotechnical Investigation consisted of the following tests. Five (5) samples were submitted to Maxxam Analytics on November 15, 2010 under chain of custody number 572722. The samples were submitted for the parameters of redox potential, resistivity, sulphate, chloride, and pH. The testing results were received on November 19, 2010, under report number BOG4161.

### *5.2 Geotechnical Testing*

The geotechnical testing component of this Geotechnical Investigation consisted of the following tests. Unit weight testing was performed on a total of eight (8) rock cores. Moisture Content testing was performed on a total of eight (8) rock cores.

## 6.0 SUBSOIL CONDITIONS

The soils encountered within the nine (9) test pit, and eight (8) borehole locations, were found to be relatively consistent. In general, the test locations had organic topsoil overlying a stiff silty clay and shallow limestone bedrock. The findings of the fieldwork are in general agreement to the depth investigated with the published geology this area.

The subsoil encountered at each test location is presented attached as *Table 1,* at the end of this report. The subsoil encountered at each borehole is presented in the *Borehole Logs* attached as *Enclosure Nos.: 1 to 8*. Each soil type is described in the following sections. The *Results of Environmental Laboratory Testing* for each of the submitted samples is presented as *Table 2*, at the end of this report.

### *6.1 Overburden*

In all test locations, a surficial covering of topsoil was observed. This topsoil is described as dark brown and predominantly organic. Traces of limestone fragments were observed within this layer. The thickness of this topsoil layer ranged from approximately 0.2m in locations TP-1, TP-3, TP-4, TP-5, and BH-7, to approximately 0.6m in locations BH-1, BH-4, and BH-5. The topsoil descriptions found within this report, the test pit logs and borehole logs should not be used for quantity takeoffs or quality assessments.

In test pit locations TP-1, TP-2, TP-3, TP-4, TP-5 and TP-8, a native silty clay was found to be underlying the topsoil. This silty clay was of a stiff consistency. It was medium brown, and was recovered in a moist condition. In some cases, cobble sized limestone fragments were observed within this soil.

### *6.2 Bedrock*

Bedrock was encountered in all of the test locations for this Site. It ranged in depth from approximately 0.3m to approximately 0.6m. The bedrock at the Site appeared to consist of a heavily weathered limestone at the rock surface. Rock coring was performed in boreholes BH-2, BH-4, BH-7, and BH-8. The bedrock in these locations was found to be a light to medium grey micritic limestone cut by numerous stylolites. In all of these locations the bedrock was found to be poor quality and heavily weathered within the upper 1.0m to 1.5m. In this heavily weathered zone, horizontal to sub-vertical joints were observed, with some containing clay infill. Below these depths, the rock in the cores became fair to good quality.

Unit weight testing was performed on a total of nine (9) intact rock samples. The unit weight was found to range from approximately 26.3 kN/m$^3$ to approximately 27.8 kN/m$^3$. Point load testing was performed on a total of nine (9) intact rock cores. The uniaxial compressive strength was found to range from approximately 134.6 MPa to approximately 172.3 MPa. It must be emphasized that the lab results were performed in individual samples of intact bedrock, and would not reflect the properties of a weathered rock mass with occasional clay infill.

## 7.0 GROUNDWATER CONDITIONS

Groundwater infiltration was not observed within any of the test pit locations. Due to the use of wet coring techniques in the borehole fieldwork program, it was impractical to make ground water observations in the open holes.

A review of the Ontario Well Registry data indicates that the water levels in this area range from approximately 5.0m to 10m below the ground surface.

Groundwater tables are subject to seasonal fluctuations and precipitation events.

## 8.0 DISCUSSION AND RECOMMENDATIONS

### *8.1 Project Description*

Based on the information provided by the Client's *Request for Quotation* (Ref No: 2010-06-10, dated June 10, 2010) and the "*Aerial / Plan View PV System Layout*" (Canadian Solar Solution Inc., Ref No: O1C1-2010-0067, dated October 13, 2010), the proposed project scope is summarized as follows:

- Design forces for solar panel foundations are considered to be 9 kN horizontally, at a height of 1.2m above the ground surface; 23 kN vertically in the downward direction; 9 kN vertically in the upward direction;
- The tolerable movement of the foundation systems are 50 mm in the horizontal direction or a maximum settlement of 25 mm in the vertical direction;
- Various concrete slabs for electrical equipment pads to be installed on grade;
- A network of connecting underground electrical circuits installed in open cut trenches to a maximum depth of 1.8m; and
- A network of connecting gravel access roads across the Site.

If any of these assumptions are incorrect or these facts change through the design or construction process, **Inspec-Sol** should be notified and retained to assess the impact of these changes from a geotechnical standpoint.

### *8.2 General Considerations*

Based upon the results of the Geotechnical Investigation and assuming that they are representative of the soil conditions of this Site, comments and recommendations are offered in the following sections for the solar panels and related infrastructure.

The most significant geotechnical considerations for the design and construction of the proposed project are related to the following:

- Frost protection for solar panel foundation elements; and
- Allowances should be considered for excavation and over-excavation to account for poor quality weathered bedrock.

### *8.3 Site Preparation*

It is expected that Site preparation for access roads and foundation areas will consist of the removal of fills, topsoil, root zone, any previously disturbed soils, and heavily weathered bedrock to expose the native undisturbed soil or rock. Once exposed, native soil subgrades should be assessed by geotechnical personnel by means of proof rolling under heavy construction equipment, to look for local anomalies or soft spots. The overburden on this Site is considered to be sensitive to disturbance by construction traffic as well as increases in moisture content. The near surface overburden may be wet and lower in strength in the spring thaw period, and fall wet season. This may result in the requirement for over-excavation of unsuitable soil, rather than if construction was completed in dry summer periods. The Owner and Contractor should make allowances for this type of "wet soil" issue.

The presence of shallow bedrock and bedrock outcrops will create problems for equipment carrying out the site clearing. Rock removal methods may be required which will be expensive, and also, it may be difficult to achieve a level grade. In foundation more so than access roadways, it may be requires to remove rock if the fractures contain soft or organic soils.

An adequate ditching and pumping system may be necessary in order to collect surface run-off or shallow groundwater, and to provide stable working conditions.

### *8.4 Excavation and Dewatering*

Excavations for the structures and site services for this project are expected to extend to a maximum depth of approximately 2.0m below the existing surface grade.

All excavations should be completed and maintained in accordance with the Occupational Health and Safety Act (OHSA) requirements. Based on the results of the Geotechnical Investigation, the native overburden encountered within the expected excavation depths are considered to be a "Type 2 Soil", as defined in the OHSA Regulations for Construction. The bedrock would be considered to be a "Type 1 Soil"

Weathered or fractured limestone bedrock encountered during excavation may require pneumatic or hydraulic breakers such as hoe rams or heavy excavation equipment equipped for rock excavation. Controlled blasting techniques may be used, subject to the laws and blasting restrictions that are in effect for the area.

Based on the weathered and poor quality condition of the near-surface bedrock noted during the fieldwork, it is recommended that the Owner and Contractor make an allowance for the over-excavation of the heavily weathered bedrock surface.

Water seepage will require adequate handling so as to minimize interference with construction.

## *8.5 Solar Panel Foundations*

Based on the project description in *Section 8.1*, the foundation options that the Client typically considers consist of: either pad foundation systems founded on competent soil or bedrock; or drilled piers advanced into bedrock. Designers and Contractors should consider the descriptions contained within this report carefully to assess the advantages and suitability of each foundation system.

### *8.5.1 Pad Foundations Systems*

The recommended design bearing capacity for pad foundations founded on suitable weathered bedrock zone is 300 kPa (factored) under Ultimate Limit States (ULS) conditions. Suitable weathered rock would be free of organic soil or soft to stiff inorganic soils within the fractures. There is no corresponding Serviceability Limit States (SLS) value for bedrock as settlement under the ULS condition is expected to be nil (0).

The design bearing capacity for pad foundations founded on deeper good quality bedrock, below the upper weathered zone is 2500 kPa (factored) under Ultimate Limit States (ULS) conditions subject to field verification. Footing sizes are expected to be in the order of 1.0m to 2.0m. There is no corresponding Serviceability Limit States (SLS) value for bedrock as settlement under the ULS condition is expected to be nil (0). The quality of the bedrock will need to be proven on Site by means of “rock probing”. A small, approximately 50 mm diameter, hole should be drilled to a depth of approximately 1.5m below the founding elevation. The sides of this hole should be scratched with a steel rod by geotechnical personnel to look for mud seams or significant fractures. The location of these “rock probes” will be a site decision based on the nature of the exposed rock subgrade.

#### *8.5.2 Drilled Pier Foundation Systems*

It is understood that the Client is considering the use of drilled pier foundation systems, which are socketed into bedrock to support the solar panel arrays.

If used, drilled pier foundation systems should be installed to a depth such that the toe of the pier is founded below the frost zone, which is considered to be 1.5m for unheated or isolated structures in this area.

The recommended design bearing capacity for pier foundations founded in the upper weathered bedrock zone is 300 kPa (factored) under Ultimate Limit States (ULS) conditions. There is no corresponding Serviceability Limit States (SLS) value for bedrock as settlement under the ULS condition is expected to be nil (0).

The design bearing capacity for pier foundations founded in the deeper good quality bedrock, below the upper weathered zone is 2500 kPa (factored) under Ultimate Limit States (ULS) conditions. There is no corresponding Serviceability Limit States (SLS) value for bedrock as settlement under the ULS condition is expected to be nil (0).

**Inspec-Sol** should be retained to review the final design prior to the start of construction.

#### *8.5.3 Resisting Uplift Forces*

Regardless of the foundation system selected, uplift capacities are assumed to be resisted by the dead weight of the foundation and the overlying soil. These comments are based on our understanding that design uplift is approximately 9 kN.

**Table 3: Suggested Parameters for Resisting Foundation Uplift**

| GEOTECHNICAL PARAMETER | RECOMMENDED DESIGN VALUE |
|---|---|
| Granular Fill compacted and placed on top of footing [$\gamma$] | 21.5 kN/$m^3$ |
| Concrete Unit Weight [$\gamma$] | 24.5 kN/$m^3$ |

If increased uplift capacities are required, this may be achieved with the use of mechanical-type or grouted rock anchors. Mechanical-type anchor design should be based upon manufacturer's recommendations.

Grouted rock anchors may be designed based on a frictional stress between grout and the limestone bedrock. Based upon typical published values and conservative approach, we recommend that a conservative allowable working stress value of 690 kPa be utilized to calculate the length of the required bond zone for grouted rock anchors. For mechanical-type anchors, manufacturer's data should be reviewed. The bond zone must be entirely within "sound bedrock" which is below the weathered zone. An allowance for a weathered rock zone of 1.5m in each hole should be incorporated.

For designing under the Limit State Design (LSD), designers may take the approach that working stress value is approximately equivalent to the SLS value. Based upon typical published values, the ULS values may be approximately 1.5 MPa to more than 2 MPa. As per the Canadian Foundation Engineering Manual (CFEM-2006), a resistance factor of 0.3 should be applied to this empirical ULS. Higher stress values may be available; however performance load testing in the field will be required to prove the capacities. In this case, a resistance factor of 0.4 should be applied as per CFEM-2006.

In order to mobilize the shear stress, the load at the top of the anchors must be properly transferred to the bottom to keep the grout in compression and ensure proper performance, and therefore a "free length" is required through the overburden and the weathered rock zone.

The mass of rock mobilized by a rock anchor may be assumed to be based upon a $60^{o}$ cone drawn up from a point located at the lower one-third point of the anchor shaft bond zone and spaced such that the theoretical cones do not overlap. Designers should review the spacing of anchors and take into account of any overlapping cones (i.e. avoid doubling-up on rock mass calculations for overlapping cones).

The bulk unit weight of limestone bedrock, according to the laboratory testing, was found to range from approximately 26.3 $kN/m^3$ up to 27.8 $kN/m^3$. It should be noted however, that this is representative of the good quality limestone component of the rock mass only. In reality, the rock mass is heavily weathered near the surface and has significant soil infill in the joints.

Based on these observations, it is recommended that the bulk and submerged unit weight design values be considered as 24 kN/m$^3$ and 14 kN/m$^3$, respectively.

#### *8.5.4 Frost Protection of Foundations*

##### *8.5.4.1 Protection of Bearing Soils*

The native soils found on Site, and the heavily weathered bedrock surface, are considered to be frost susceptible. All bearing surfaces will require a minimum of 1.5m of soil cover, or an equivalent insulation detail. **Inspec-Sol** should be retained to review a specific insulation detail if this option is chosen.

##### *8.5.4.2 Frost Adhesion Uplift Forces*

Frost adhesion and resulting forces will vary significantly depending on soil types, moisture content, sources of fill moisture, freezing days, freeze-thaw conditions etc. Uplift forces are typically not provided for foundation elements, but rather good practice is to incorporate materials or foundation shapes that minimize or prevent impacts of frost forces. Therefore, it is recommended that the foundation elements be backfilled with a coarse-grained non-frost susceptible soil. One example would be an OPSS Granular 'B', Type I. Alternatively, we recommend that the use of a bond break be incorporated into the design of the foundation system to prevent frost adhering to foundation elements, such as grease or plastic sheathing.

If designers wish to not use provisions for minimizing frost adhesion but rather design for uplift forces then we would refer the designers to Section 13.5.1 of the Canadian Foundation Engineering Material (CFEM) or other documented references.

#### *8.5.5 Seismic Site Classification*

Based on the field programs that were undertaken as part of this Geotechnical Investigation, shallow bedrock was encountered within the sampling depths. At this point there is enough information available to assign a "Site Class C" according to Table 4.1.8.4.A of the Ontario Building Code (OBC-2006). However, we are of the understanding that this project is not governed by Part 4 of OBC-2006 and this is not necessary in the design. If a higher Site Class is beneficial, please inform **Inspec-Sol** if this is required so we can discuss the available options.

### *8.5.6 Chemical Results of Soil Analysis*

The results of the chemical testing of soils included in the scope of work is intended for design purposes to initially assess for evidence of corrosive soil environments for buried metal or concrete. The chemical results should be reviewed based upon application and location of buried element.

In terms of buried metals and interpretation of the data, we would suggest that designers review the data and if they require assistance then **Inspec-Sol** may be able to be retained to provide further assistance but it may depend upon the application.

In terms of buried concrete, the results from the five (5) samples tested indicate they are below the requirements for moderate potential for sulphate attack of buried concrete as per Table 3 of the CSA A23.1 standard and designers/contractors should produce concrete mixes that comply with the standard. If further assistance or investigation is required, then contact **Inspec-Sol** for retaining for further assistance.

### *8.5.7 Engineered Fill*

The fill operations for Engineered Fill, if necessary, on this Site must satisfy the following criteria:

- Engineered Fill must be placed under continuous supervision of the Geotechnical Engineer. Prior to placing any Engineered Fill, all unsuitable fill materials must be removed, and the subgrade proof rolled, and approved. Any deficient areas should be repaired;
- Prior to the placement of Engineered Fill, the source or borrow areas for the Engineered Fill must be evaluated for its suitability. Samples of proposed fill material must be provided to the Geotechnical Engineer and tested in the geotechnical laboratory for Standard Proctor Maximum Dry Density (SPMDD) and grain size, prior to approval of the material for use as Engineered Fill. The Engineered Fill must consist of environmentally suitable soils (as per industry standard procedures of federal or provincial guidelines/regulations), free of organics and other deleterious material (building debris such as wood, bricks, metal, and the like), compactable, and of suitable moisture content so that it is within -2% to

+0.5% of the Optimum Moisture as determined by the Standard Proctor Test. Imported granular soils meeting the requirements of Granular 'A';

- The Engineered Fill must be placed in maximum loose lift thicknesses of 0.2m. Each lift of Engineered Fill must be compacted with a heavy roller to 100% of its SPMDD; and
- Field density tests must be taken by the Geotechnical Engineer, on each lift of Engineered Fill. Any Engineered Fill, which is tested and found to not meet the specifications, shall be either removed or reworked and retested.

## *8.6 Construction of Underground Services*

### *8.6.1 Bedding and Cover*

The following are recommendations for service trench bedding and cover materials:

- Bedding for buried utilities should be OPSS Granular 'A' and placed in accordance with pertinent Ontario Provincial Standard Drawings (OPSD);
- Use of clear 19 mm stone is not recommended for use as bedding;
- The cover material should be a sand material or Granular 'A' and the dimensions should comply with pertinent OPSD standards;
- The bedding material and cover materials should be compacted as per OPSS 501 and 514 and to at least 95% of its SPMDD; and
- Compaction equipment should be used in such a way that the utility pipes are not damaged during construction.

### *8.6.2 Service Trench Backfill*

Backfill above the cover for buried utilities should be in accordance with the following recommendations:

- For service trenches under access roads, the backfill should be placed and compacted in uniform thickness compatible with the selected compaction equipment and not thicker than 200 mm. Each lift should be compacted to a minimum of 95% of its SPMDD;
- The backfill placed in the upper 300 mm below the access road subgrade elevation should be compacted to a minimum of 100% of its SPMDD;

- To reduce the potential for differential settlement and frost heave, the selected backfill materials should reasonably match the existing soil profile within the frost penetration zone (1.5m below finished grade). Alternatively, if imported backfill, including granular materials, are used then the excavation sides should have frost tapers as per OPSD 800 series which essentially indicates that there should be a backslope of 10H:1V from the bedding grade to the finished grade;
- If considered, rock fill should be reviewed and approved by a Geotechnical Engineer prior to placement. In service trenches slightly larger rock material may be used, (i.e. < 300 mm nominal diameter type material). Material greater than 300 mm diameter may be crushed down to an acceptable size range prior to usage. It should be placed above pipe cover materials and as it is brought up to its final grade, it should have more fines added and the final surface be chinked to ensure a closed face. This surface should be reviewed by the Geotechnical Engineer representative and if too coarse or open due to segregation then a geotextile covering may be required or where segregation exists, it may be necessary to mix with an OPSS Granular 'A'. In such instances it is preferable to mix the finer material in with the rock fill thoroughly to the approval of the Geotechnical Engineer prior to placement;
- Rock fill material should be placed in 250 mm ± 50 mm lifts; and
- Where rock fill is used for utility backfill, frost tapers and proper transition detailing should be in accordance to OPSS 205.

### *8.7 Construction of Gravel Access Roads*

Gravel access roads are expected to be constructed over the undisturbed silty clay subgrade or on the exposed bedrock. In order to prepare the Site for the roadway structure, it is recommended that the topsoil, fill and disturbed soils be stripped and the exposed subgrade be, proof rolled with heavy construction equipment to identify "soft spots" or local anomalies. Any areas where rutting or appreciable deflection is noted should be sub-excavated and replaced with suitable fill, and use of geotextiles may be warranted for strength improvement. Any fill used to raise the grade below future access road structure should be considered Engineered Fill and should be treated as such.

The near surface overburden may be wet and lower in strength in the spring thaw period, and fall wet season. This may result in the requirement for over-excavation of unsuitable soil, rather than if construction was completed in dry summer periods. The Owner and Contractor should make allowances for this type of “wet soil” issue.

The gravel sections described in *Table 4* below are general recommendations for areas subjected to parking lot and access road traffic. Alternative designs would require additional testing and analysis.

**Table 4: Recommended Access Road Structure**

| Pavement Layer | Light Duty (Parking Areas) | Heavy Duty (Travelled Roadways) | Access Roads on Rock Fill Subgrade |
|---|---|---|---|
| Granular A Base | 200 mm | 250 mm | 300 mm |
| Granular B Type II Subbase | 300 mm | 350 mm | - |

It is possible to use the previously excavated rock fill as subbase for the roadways. If used, rock fill should be reviewed and approved by a Geotechnical Engineer prior to placement. Material greater than 150 mm diameter must be crushed down to an acceptable size range prior to usage. Rock fill surface must be verified to be chinked and present a closed face to prevent migration of fines. If the exposed surface is too coarse and open then a layer of geotextile may be required. This surface should be reviewed by the Geotechnical Engineer representative and if too coarse or open due to segregation then it may be necessary to mix with an OPSS Granular ‘A’. In such instances it is preferable to mix the finer material in with the rock fill thoroughly to the approval of the Geotechnical Engineer prior to placement.

Drainage of the granular layers is important. The surface of each layer of the access road section should be provided with a suitable cross fall (approximately 2%) to prevent water from ponding on the surface and beneath the layers. Surface runoff should be directed into ditches.

Sufficient field-testing should be carried out during construction to assess compaction of each lift of the granular courses. All granular base and subbase course materials should be compacted to 100% of their SPMDD.

The recommended access road structures provided within this report are considered sufficient for the expected end-use conditions only, which includes light vehicular traffic and occasional maintenance vehicles. These granular thicknesses may be required to be increased to properly support heavy equipment during construction period. Regular maintenance will be required to achieve maximum life expectancy.

## 9.0 CONSTRUCTION FIELD REVIEW

The discussion and recommendations provided within this report are based upon our current understanding of the project. **Inspec-Sol** requests to be retained to review the structural plans and specifications once they become available to verify that the recommendations within this report have been adequately addressed, and to look for obvious geotechnical problems.

The recommendations provided in this report are also based on an adequate level of construction monitoring being conducted during the construction phase of the proposed development. Due to the nature of the proposed development, an adequate level of construction monitoring is considered to be as follows:

- Prior to construction of footings, the exposed footing subgrade should be examined by a Geotechnical Engineer or a qualified technologist acting under the supervision of a Geotechnical Engineer, to assess whether the subgrade conditions correspond to those encountered in the test pit and borehole locations, and that the recommendations provided in this report have been implemented;
- The use of Engineered Fill, if necessary should be monitored on a full-time basis by a qualified engineering technologist;
- The materials proposed to be used as backfill or granular courses should be submitted to a Geotechnical Engineer for testing and approval prior to use;
- Backfilling operations should be conducted in the presence of a qualified technologist to ensure that proper material is employed and specified compaction is achieved;
- Testing of concrete strength should be conducted by a qualified technologist to ensure the specifications for the project have been met;
- A Site-specific program of Quality Control or Quality Assurance should be developed to verify the ongoing installation of the drilled pier foundation systems; and
- Several compression and tension load-testing programs should be undertaken at the outset of the Project to ensure that design uplift and bearing capacities are being achieved.

## 10.0 REPORT CONDITIONS AND LIMITATIONS

This report is intended solely for Canadian Solar Solutions Inc. and the other parties explicitly identified within the report. It is prohibited for use by others without **Inspec-Sol**'s prior written consent. This report is considered **Inspec-Sol**'s professional work product and shall remain the sole property of **Inspec-Sol**. Any unauthorized reuse, redistribution of or reliance on the report shall be at the Client and recipient's sole risk, without liability to **Inspec-Sol**. Client shall defend, indemnify and hold **Inspec-Sol** harmless from any liability arising from or related to Client's unauthorized distribution of the report. No portion of this report may be used as a separate entity; it is to be read in its entirety and shall include all supporting drawings and appendices.

The recommendations made in this report are in accordance with our present understanding of the project, the current site use, ground surface elevations and conditions, and are based on the work scope approved by the Client and described in the report. The services were performed in a manner consistent with that level of care and skill ordinarily exercised by members of geotechnical engineering professions currently practicing under similar conditions in the same locality. No other representations, and no warranties or representations of any kind, either expressed or implied, are made. 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.

All details of design and construction are rarely known at the time of completion of a geotechnical study. The recommendations and comments made in the study report are based on our subsurface investigation and resulting understanding of the project, as defined at the time of the study. We should be retained to review our recommendations when the drawings and specifications are complete. Without this review, **Inspec-Sol** will not be liable for any misunderstanding of our recommendations or their application and adaptation into the final design.

By issuing this report, **Inspec-Sol** is the Geotechnical Engineer of record. It is recommended that **Inspec-Sol** be retained during construction of all foundations and during earthwork operations to confirm the conditions of the subsoil are actually similar to those observed during our study. The intent of this requirement is to verify that conditions encountered during construction are consistent with the findings in the report and that inherent knowledge developed as part of our study is correctly carried forward to the construction phases.

It is important to emphasize that a soil investigation is, in fact, a random sampling of a site and the comments included in this report are based on the results obtained at the nine (9) test pit and eight (8) borehole locations only. The subsurface conditions confirmed at these seventeen (17) test locations may vary at other locations. Soil and groundwater conditions between and beyond the test locations may differ both horizontally and vertically from those encountered at the test locations and conditions may become apparent during construction, which could not be detected or anticipated at the time of our investigation. Should any conditions at the Site be encountered which differ from those found at the test locations, we request that we be notified immediately in order to permit a reassessment of our recommendations. If changed conditions are identified during construction, no matter how minor, the recommendations in this report shall be considered invalid until sufficient review and written assessment of said conditions by **Inspec-Sol** is completed.

**INSPEC-SOL INC.**

Shane Dunstan, B.A.Sc., E.I.T.

Joseph B. Bennett, P. Eng.

SD/nc

INSPEC-SOL INC.
179 Colonnade Rd., Suite 400
Ottawa, ON K2E 7J4
Tel.:(613) 727-0895 Fax: (613) 727-0581

Reference No.: **T020822-A2**
Client: **Canadian Solar Solutions Inc.**
Project: **UPPER CANADA SOLAR 02, Elizabethtown-Kitley, ON**

***Table No.:1 Test Pit Logs***

| Location | Depth (m) | | | Description |
|---|---|---|---|---|
| TP-1 | 0.0 | - | 0.2 | Topsoil over silty clay, dark brown, moist (organic stained) |
| | 0.2 | - | 0.5 | Silty clay, stiff, medium brown, moist, limestone fragments |
| | | 0.5 | | End of test pit on bedrock |
| TP-2 | 0.0 | - | 0.3 | Topsoil over silty clay, dark brown, moist, limestone fragments (organic stained) |
| | | 0.3 | | Fractured limestone layer |
| | 0.3 | - | 0.6 | Silty clay, stiff, medium brown, moist |
| | | 0.6 | | End of test pit on bedrock |
| TP-3 | 0.0 | - | 0.2 | Topsoil over silty clay, dark brown, moist (organic stained) |
| | 0.2 | - | 0.6 | Silty clay, stiff, medium brown, moist |
| | | 0.6 | | End of test pit on bedrock |
| TP-4 | 0.0 | - | 0.2 | Topsoil over silty clay, dark brown, moist (organic stained) |
| | 0.2 | - | 0.4 | Silty clay, stiff, medium brown, moist |
| | | 0.6 | | End of test pit on bedrock |
| TP-5 | 0.0 | - | 0.2 | Topsoil over silty clay, dark brown, moist (organic stained) |
| | 0.2 | - | 0.5 | Silty clay some limestone fragments, stiff, medium brown, moist |
| | | 0.6 | | End of test pit on bedrock, some surface water infiltiration |
| TP-6 | 0.0 | - | 0.3 | Topsoil over silty clay, dark brown, moist (organic stained) |
| | | 0.3 | | End of test pit on bedrock, some silty clay infill in surficial fractures |
| TP-7 | 0.0 | - | 0.3 | Topsoil over silty clay, dark brown, moist (organic stained) |
| | | 0.3 | | End of test pit on bedrock |
| TP-8 | 0.0 | - | 0.3 | Topsoil over silty clay, dark brown, moist (organic stained) |
| | 0.3 | - | 0.5 | Silty clay, stiff, medium brown, moist |
| | | 0.5 | | End of test pit on bedrock |
| TP-9 | 0.0 | - | 0.3 | Topsoil over silty clay, dark brown, moist, limestone fragments (organic stained) |
| | | 0.3 | | End of test pit on bedrock |

INSPEC-SOL INC.
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Tel.:(613) 727-0895   Fax: (613) 727-0581

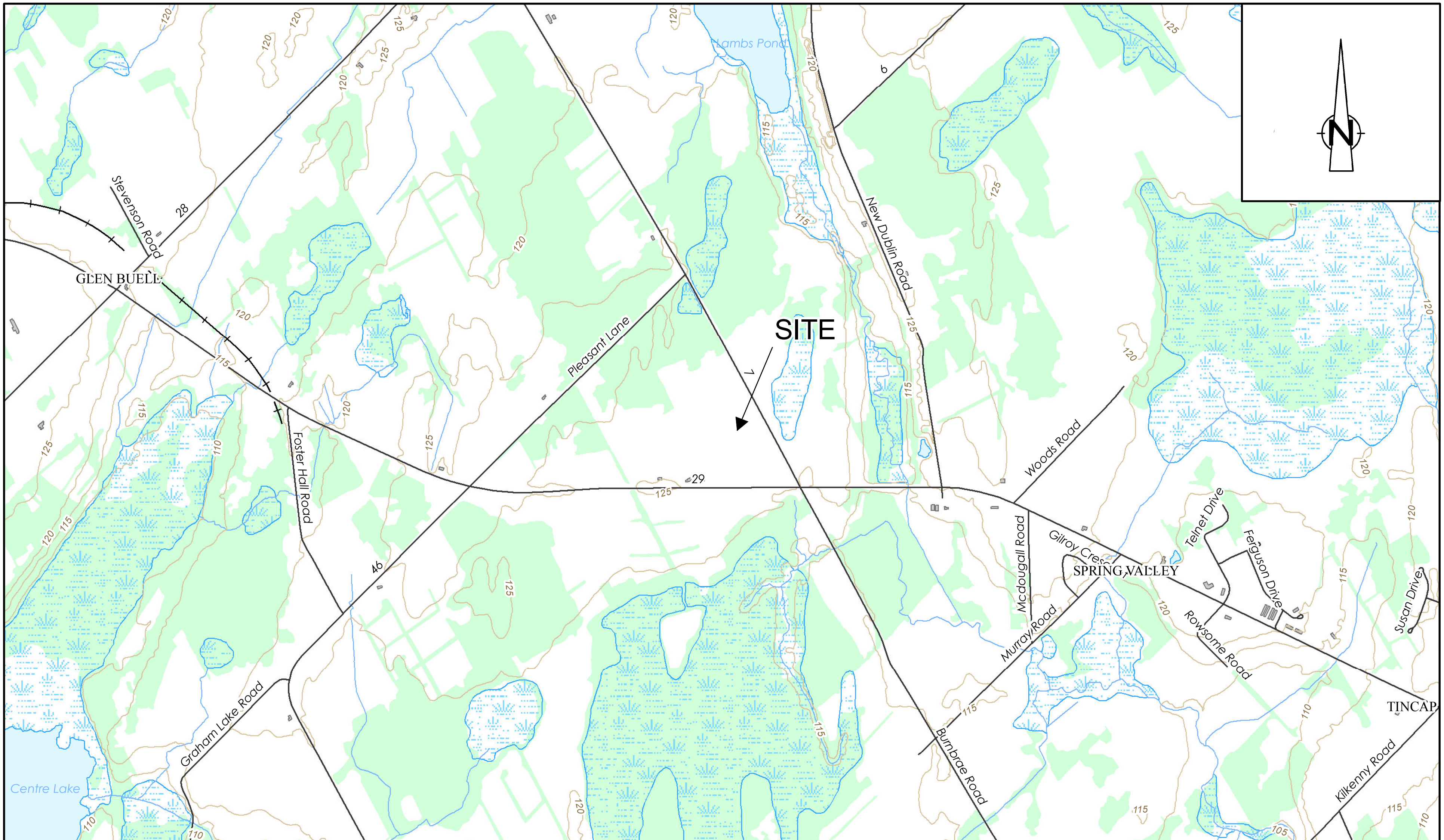
Reference No.: **T020822-A1**
Client: **Canadian Solar Solutions Inc.**
Project: **UPPER CANADA SOLAR 02, Elizabethtown-Kitley, ON**

***Table No.:2  Results of Laboratory Testing***

| **Sample No.** | **TP2** | **TP3** | **TP5** | **TP7** | **TP-8** |
|---|---|---|---|---|---|
| Depth (m) | 0.0 - 0.3 | 0.3 - 0.6 | 0.2 - 0.5 | 0.0 - 0.3 | 0.3 - 0.5 |
| Soil Type | Topsoil | Silty Clay | Silty Clay | Topsoil | Silty Clay |
| **Parameter** | | | | | |
| pH | 6.66 | 6.83 | 7.36 | 7.43 | 7.01 |
| Chloride [$Cl^-$] (ug/g) | ND | 44 | ND | 26 | ND |
| Sulphate [$SO_4^-$] (ug/g) | ND | 79 | ND | ND | ND |
| Redox Potential (mV) | 188 | 206 | 212 | 212 | 195 |
| Conductivity (umho/cm) | 58 | 69 | 169 | 160 | 66 |

# D R A W I N G S

**Site Location Map**
**Test Pit and Borehole Location Plan**




Source: MNR NRVIS, 2010. Produced by CRA under licence from Ontario Ministry of Natural Resources, © Queen's Printer 2010 Datum: NAD 83 Projection: UTM Zone 18

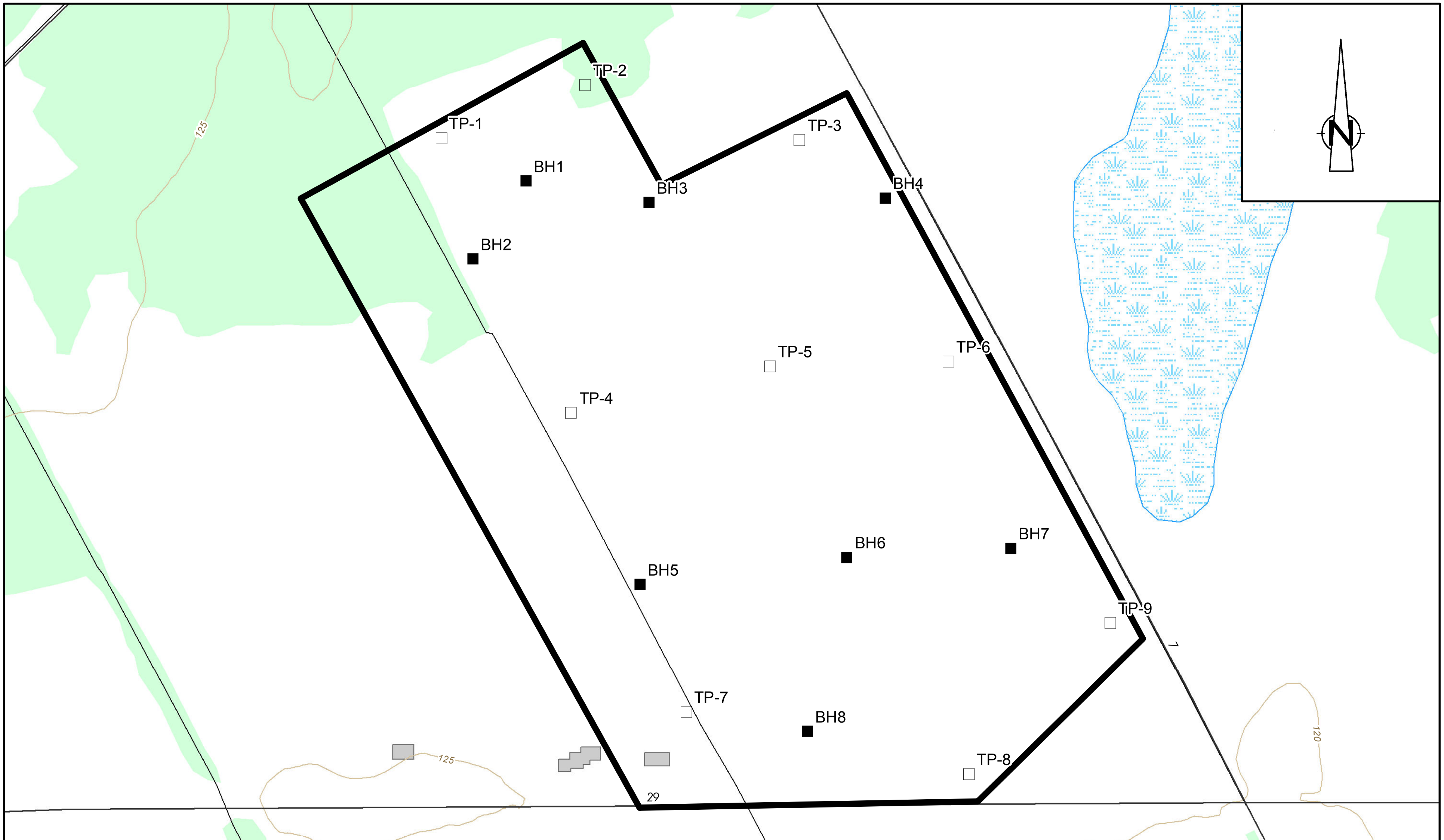
# SITE LOCATION MAP

## GEOTECHNICAL INVESTIGATION
## PROPOSED SOLAR FARM
## BROCKVILLE, ONTARIO

*Dwg. No. T020822-A2-1*

INSPEC-SOL

T020822-A2(001)GIS-OT001 February 01, 2011




Source: MNR NRVIS, 2010. Produced by CRA under licence from Ontario Ministry of Natural Resources, © Queen's Printer 2010 Datum: NAD 83 Projection: UTM Zone 18

# BOREHOLE AND TEST PIT LOCATION PLAN

GEOTECHNICAL INVESTIGATION
PROPOSED SOLAR FARM
BROCKVILLE, ONTARIO
*Dwg. No. T020822-A2-2*

INSPEC-SOL

T020822-A2(001)GIS-OT002 February 02, 2011

# ENCLOSURES

**Borehole Logs**
**Enclosure Nos.: 1 to 8**

INSPEC·SOL

**BOREHOLE No.: BH-1**

**ELEVATION:**

# BOREHOLE LOG



CLIENT: Canadian Solar Solutions Inc.

PROJECT: Geotechnical Investigation

LOCATION: 5061 County Road 29, Elizabethtown-Kitley, Ontario

DESCRIBED BY: D. Hill CHECKED BY: S. Dunstan

DATE (START): January 6, 2011 DATE (FINISH): January 6, 2011

**LEGEND**

- SS Split Spoon
- ST Shelby Tube
- RC Rock Core
- Water Level
- Water content (%)
- Atterberg limits (%)
- ● N Penetration Index based on Split Spoon sample
- • N Penetration Index based on Dynamic Cone sample
- △ Cu Shear Strength based on Field Vane
- □ Cu Shear Strength based on Lab Vane
- S Sensitivity Value of Soil
- ▲ Shear Strength based on Pocket Penetrometer

| SCALE | | STRATIGRAPHY | | SAMPLE DATA | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| Depth BGS | Elevation (m) | Stratigraphy | DESCRIPTION OF SOIL AND BEDROCK | State | Type and Number | Recovery | OVC | Penetration Index / RQD | SCALE FOR TEST RESULTS (50kPa 100kPa 150kPa 200kPa; 10 20 30 40 50 60 70 80 90) |
| meters | | | GROUND SURFACE | | | % | ppm | N | |
| 0.0–0.5 | | | **TOPSOIL** - sandy, dark brown, moist. | SS | SS1 | 45 | | 9 | ● |
| 0.6 | | | Borehole terminated at 0.6 m on limestone bedrock. | | | | | | |
| 1.0 | | | | | | | | | |
| 1.5 | | | | | | | | | |
| 2.0 | | | | | | | | | |
| 2.5 | | | | | | | | | |
| 3.0 | | | | | | | | | |
| 3.5 | | | | | | | | | |
| 4.0 | | | | | | | | | |
| 4.5 | | | | | | | | | |

NOTES:

REFERENCE No.: T020822-A2 ENCLOSURE No.: 2

INSPEC·SOL

**BOREHOLE No.: BH-2**

**ELEVATION:**

# BOREHOLE LOG



CLIENT: Canadian Solar Solutions Inc.

PROJECT: Geotechnical Investigation

LOCATION: 5061 County Road 29, Elizabethtown-Kitley, Ontario

DESCRIBED BY: D. Hill CHECKED BY: S. Dunstan

DATE (START): January 6, 2011 DATE (FINISH): January 6, 2011

**LEGEND**

- SS Split Spoon
- ST Shelby Tube
- RC Rock Core
- Water Level
- Water content (%)
- Atterberg limits (%)
- ● N Penetration Index based on Split Spoon sample
- • N Penetration Index based on Dynamic Cone sample
- △ Cu Shear Strength based on Field Vane
- □ Cu Shear Strength based on Lab Vane
- S Sensitivity Value of Soil
- ▲ Shear Strength based on Pocket Penetrometer

| Depth BGS (meters) | Elevation (m) | DESCRIPTION OF SOIL AND BEDROCK | State | Type and Number | Recovery (%) | OVC (ppm) | Penetration Index / RQD (N) |
|---|---|---|---|---|---|---|---|
| | | GROUND SURFACE | | | | | |
| 0.0 | | **TOPSOIL** - sandy, dark brown, moist. | SS | SS1 | 83 | | R |
| 0.35 | | **LIMESTONE** - light grey, micritic, cut by occasional stylolites, sub-horizontal fractures, heavily weathered, poor quality. Calcite banding. | RC | RC1 | 100 | | 25 |
| 1.7 | | Becoming fair quality, moderately weathered. Vertical joint. | RC | RC2 | 100 | | 75 |
| 3.3 | | Borehole terminated at 3.3 m in limestone bedrock. | | | | | |

Depth scale: 0.5, 1.0, 1.5, 2.0, 2.5, 3.0, 3.5, 4.0, 4.5

SCALE FOR TEST RESULTS: 50kPa, 100kPa, 150kPa, 200kPa; 10, 20, 30, 40, 50, 60, 70, 80, 90

NOTES:

REFERENCE No.: T020822-A2 ENCLOSURE No.: 3

**BOREHOLE No.: BH-3**

**ELEVATION:**

# BOREHOLE LOG



CLIENT: Canadian Solar Solutions Inc.

PROJECT: Geotechnical Investigation

LOCATION: 5061 County Road 29, Elizabethtown-Kitley, Ontario

DESCRIBED BY: D. Hill CHECKED BY: S. Dunstan

DATE (START): January 6, 2011 DATE (FINISH): January 6, 2011

**LEGEND**

- SS Split Spoon
- ST Shelby Tube
- RC Rock Core
- Water Level
- Water content (%)
- Atterberg limits (%)
- N Penetration Index based on Split Spoon sample
- N Penetration Index based on Dynamic Cone sample
- Cu Shear Strength based on Field Vane
- Cu Shear Strength based on Lab Vane
- S Sensitivity Value of Soil
- Shear Strength based on Pocket Penetrometer

| SCALE | | STRATIGRAPHY | | SAMPLE DATA | | | | | SCALE FOR TEST RESULTS |
|---|---|---|---|---|---|---|---|---|---|
| Depth BGS | Elevation (m) | Stratigraphy | DESCRIPTION OF SOIL AND BEDROCK | State | Type and Number | Recovery | OVC | Penetration Index / RQD | 50kPa 100kPa 150kPa 200kPa; 10 20 30 40 50 60 70 80 90 |
| meters | | | GROUND SURFACE | | | % | ppm | N | |
| 0.0–0.5 | | | **TOPSOIL** - sandy, dark brown, moist. | X | SS1 | 38 | | | |
| 0.5 | | | Borehole terminated at 0.5 m on limestone bedrock. | | | | | | |
| 1.0 | | | | | | | | | |
| 1.5 | | | | | | | | | |
| 2.0 | | | | | | | | | |
| 2.5 | | | | | | | | | |
| 3.0 | | | | | | | | | |
| 3.5 | | | | | | | | | |
| 4.0 | | | | | | | | | |
| 4.5 | | | | | | | | | |

NOTES:

REFERENCE No.: T020822-A2 ENCLOSURE No.: 4

INSPEC•SOL

**BOREHOLE No.: BH-4**

**ELEVATION:**

**BOREHOLE LOG**

Page: 1 of 1

CLIENT: Canadian Solar Solutions Inc.

PROJECT: Geotechnical Investigation

LOCATION: 5061 County Road 29, Elizabethtown-Kitley, Ontario

DESCRIBED BY: D. Hill CHECKED BY: S. Dunstan

DATE (START): January 5, 2011 DATE (FINISH): January 5, 2011

**LEGEND**

- SS Split Spoon
- ST Shelby Tube
- RC Rock Core
- Water Level
- Water content (%)
- Atterberg limits (%)
- ● N Penetration Index based on Split Spoon sample
- • N Penetration Index based on Dynamic Cone sample
- △ Cu Shear Strength based on Field Vane
- □ Cu Shear Strength based on Lab Vane
- S Sensitivity Value of Soil
- ▲ Shear Strength based on Pocket Penetrometer

| Depth BGS (meters) | Elevation (m) | Stratigraphy | DESCRIPTION OF SOIL AND BEDROCK | State | Type and Number | Recovery (%) | OVC (ppm) | Penetration Index / RQD (N) | SCALE FOR TEST RESULTS |
|---|---|---|---|---|---|---|---|---|---|
| | | | GROUND SURFACE | | | | | | |
| 0.0–0.6 | | | **TOPSOIL** - sandy, dark brown, moist. | SS | SS1 | 29 | | 2 | ● 2 |
| 0.6–1.0 | | | | SS | SS2 | NR | | R | |
| 1.0 | | | **LIMESTONE** - light grey, micritic, cut by occasional stylolites, sub-horizontal fractures, heavily weathered, poor quality. | RC | | | | | |
| 1.5 | | | Gravel sized shatter and soil infill for 0.1 m. | RC | RC1 | 98 | | 35 | |
| 2.6 | | | Becoming fair quality, moderately weathered. | RC | RC2 | 97 | | 71 | |
| 4.2 | | | Borehole terminated at 4.2 m in limestone bedrock. | | | | | | |

SCALE FOR TEST RESULTS: 50kPa 100kPa 150kPa 200kPa; 10 20 30 40 50 60 70 80 90

Depth scale: 0.5, 1.0, 1.5, 2.0, 2.5, 3.0, 3.5, 4.0, 4.5

NOTES:

INSPEC·SOL

**BOREHOLE No.: BH-5**

**ELEVATION:**

# BOREHOLE LOG



CLIENT: Canadian Solar Solutions Inc.

PROJECT: Geotechnical Investigation

LOCATION: 5061 County Road 29, Elizabethtown-Kitley, Ontario

DESCRIBED BY: D. Hill CHECKED BY: S. Dunstan

DATE (START): January 5, 2011 DATE (FINISH): January 5, 2011

**LEGEND**

- SS Split Spoon
- ST Shelby Tube
- RC Rock Core
- Water Level
- Water content (%)
- Atterberg limits (%)
- ● N Penetration Index based on Split Spoon sample
- • N Penetration Index based on Dynamic Cone sample
- △ Cu Shear Strength based on Field Vane
- □ Cu Shear Strength based on Lab Vane
- S Sensitivity Value of Soil
- ▲ Shear Strength based on Pocket Penetrometer

| SCALE | | STRATIGRAPHY | | SAMPLE DATA | | | | |
|---|---|---|---|---|---|---|---|---|
| Depth BGS | Elevation (m) | Stratigraphy | DESCRIPTION OF SOIL AND BEDROCK | State | Type and Number | Recovery | OVC | Penetration Index / RQD |
| meters | | | GROUND SURFACE | | | % | ppm | N |
| 0.0–0.6 | | | **TOPSOIL** - sandy, dark brown, moist. | SS | SS1 | 50 | | 2 |
| 0.6 | | | Borehole terminated at 0.6 m on limestone bedrock. | | | | | |

Depth scale: 0.5, 1.0, 1.5, 2.0, 2.5, 3.0, 3.5, 4.0, 4.5

SCALE FOR TEST RESULTS: 50kPa 100kPa 150kPa 200kPa; 10 20 30 40 50 60 70 80 90

NOTES:

INSPEC·SOL

**BOREHOLE No.: BH-6**

**ELEVATION:**

# BOREHOLE LOG



CLIENT: Canadian Solar Solutions Inc.

PROJECT: Geotechnical Investigation

LOCATION: 5061 County Road 29, Elizabethtown-Kitley, Ontario

DESCRIBED BY: D. Hill CHECKED BY: S. Dunstan

DATE (START): January 5, 2011 DATE (FINISH): January 5, 2011

**LEGEND**

- SS Split Spoon
- ST Shelby Tube
- RC Rock Core
- Water Level
- ○ Water content (%)
- Atterberg limits (%)
- ● N Penetration Index based on Split Spoon sample
- • N Penetration Index based on Dynamic Cone sample
- △ Cu Shear Strength based on Field Vane
- □ Cu Shear Strength based on Lab Vane
- S Sensitivity Value of Soil
- ▲ Shear Strength based on Pocket Penetrometer

| SCALE | | STRATIGRAPHY | | SAMPLE DATA | | | | | SCALE FOR TEST RESULTS |
|---|---|---|---|---|---|---|---|---|---|
| Depth BGS | Elevation (m) | Stratigraphy | DESCRIPTION OF SOIL AND BEDROCK | State | Type and Number | Recovery | OVC | Penetration Index / RQD | 50kPa 100kPa 150kPa 200kPa; 10 20 30 40 50 60 70 80 90 |
| meters | | | GROUND SURFACE | | | % | ppm | N | |
| | | | **TOPSOIL** - sandy, dark brown, moist. | | SS1 | 72 | | R | |
| 0.5 | | | Borehole terminated at 0.5 m on limestone bedrock. | | | | | | |
| 1.0 | | | | | | | | | |
| 1.5 | | | | | | | | | |
| 2.0 | | | | | | | | | |
| 2.5 | | | | | | | | | |
| 3.0 | | | | | | | | | |
| 3.5 | | | | | | | | | |
| 4.0 | | | | | | | | | |
| 4.5 | | | | | | | | | |

NOTES:

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**BOREHOLE No.: BH-7**

**ELEVATION:**

# BOREHOLE LOG



CLIENT: Canadian Solar Solutions Inc.

PROJECT: Geotechnical Investigation

LOCATION: 5061 County Road 29, Elizabethtown-Kitley, Ontario

DESCRIBED BY: D. Hill CHECKED BY: S. Dunstan

DATE (START): January 5, 2011 DATE (FINISH): January 5, 2011

**LEGEND**

- SS Split Spoon
- ST Shelby Tube
- RC Rock Core
- Water Level
- Water content (%)
- Atterberg limits (%)
- N Penetration Index based on Split Spoon sample
- N Penetration Index based on Dynamic Cone sample
- Cu Shear Strength based on Field Vane
- Cu Shear Strength based on Lab Vane
- S Sensitivity Value of Soil
- Shear Strength based on Pocket Penetrometer

| Depth BGS (meters) | Elevation (m) | Stratigraphy | DESCRIPTION OF SOIL AND BEDROCK | State | Type and Number | Recovery (%) | OVC (ppm) | Penetration Index / RQD (N) |
|---|---|---|---|---|---|---|---|---|
| | | | GROUND SURFACE | | | | | |
| 0.0 | | | **TOPSOIL** - sandy, dark brown, moist. | SS | SS1 | 66 | | R |
| | | | **LIMESTONE** - light grey, micritic, cut by occasional stylolites, sub-horizontal fractures, good quality. | RC | RC1 | 88 | | 62 |
| 1.8 | | | Becoming fair quality, some presence of oxidation in fractures. | RC | RC2 | 100 | | 82 |
| | | | Heavily fractured for 0.4 m. | | | | | |
| 3.2 | | | Borehole terminated at 3.2 m in limestone bedrock. | | | | | |

Depth scale: 0.5, 1.0, 1.5, 2.0, 2.5, 3.0, 3.5, 4.0, 4.5

SCALE FOR TEST RESULTS: 50kPa, 100kPa, 150kPa, 200kPa; 10, 20, 30, 40, 50, 60, 70, 80, 90

NOTES:

REFERENCE No.: T020822-A2 ENCLOSURE No.: 8

INSPEC-SOL

**BOREHOLE No.: BH-8**

**ELEVATION:**

# BOREHOLE LOG



CLIENT: Canadian Solar Solutions Inc.

PROJECT: Geotechnical Investigation

LOCATION: 5061 County Road 29, Elizabethtown-Kitley, Ontario

DESCRIBED BY: D. Hill CHECKED BY: S. Dunstan

DATE (START): January 5, 2011 DATE (FINISH): January 5, 2011

**LEGEND**

- SS Split Spoon
- ST Shelby Tube
- RC Rock Core
- Water Level
- ○ Water content (%)
- Atterberg limits (%)
- ● N Penetration Index based on Split Spoon sample
- • N Penetration Index based on Dynamic Cone sample
- △ Cu Shear Strength based on Field Vane
- □ Cu Shear Strength based on Lab Vane
- S Sensitivity Value of Soil
- ▲ Shear Strength based on Pocket Penetrometer

| Depth BGS (meters) | Elevation (m) | Stratigraphy | DESCRIPTION OF SOIL AND BEDROCK | State | Type and Number | Recovery (%) | OVC (ppm) | Penetration Index / RQD (N) |
|---|---|---|---|---|---|---|---|---|
| | | | GROUND SURFACE | | | | | |
| 0.0 | | | **TOPSOIL** - sandy, dark brown, moist. | | SS1 | 63 | | R |
| 0.5 | | | **LIMESTONE** - light grey, micritic, cut by occasional stylolites, sub-horizontal fractures, poor quality, soil infill until 1.5 m below ground surface. | | RC1 | 100 | | 36 |
| 2.0 | | | Becoming fair quality. Calcite intrusions. | | RC2 | 100 | | 87 |
| 3.4 | | | Borehole terminated at 3.4 m in limestone bedrock. | | | | | |

SCALE FOR TEST RESULTS: 50kPa 100kPa 150kPa 200kPa; 10 20 30 40 50 60 70 80 90

Depth scale: 0.5, 1.0, 1.5, 2.0, 2.5, 3.0, 3.5, 4.0, 4.5

NOTES:

# APPENDICES

# A P P E N D I X  A

**Explanatory Notes for Borehole and Test Pit Logs**

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# NOTES ON BOREHOLE AND TEST PIT REPORTS

## SOIL DESCRIPTION:

Each subsoil stratum is described using the following terminology. The relative density of granular soils is determined by the standard penetration index ("N" value), while the consistency of clayey soils is measured by the value of the undrained shear strength (Cu).

**CLASSIFICATION** (UNIFIED SYSTEM)

| | | | |
|---|---|---|---|
| Clay | < 0,002mm | | |
| Silt | 0,002 to 0,075mm | | |
| Sand | 0,075 to 4,75mm | fine | 0,075 to 0,425mm |
| | | medium | 0,425mm to 2,0mm |
| | | coarse | 2,0 to 4,75mm |
| Gravel | 4,75 to 75mm | fine | 4,75mm to 19mm |
| | | coarse | 19 to 75mm |
| Cobbles | 75 to 300mm | | |
| Boulders | > 300mm | | |

**TERMINOLOGY**

| | |
|---|---|
| "traces" | 1 - 10% |
| "some" | 10 - 20% |
| adjective (silty, sandy) | 20 - 35% |
| "and" | 35 - 50% |

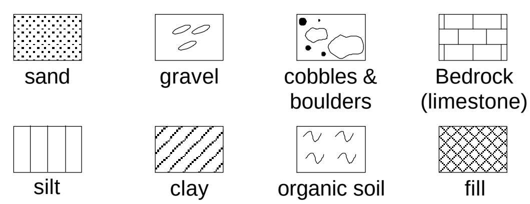
| RELATIVE DENSITY OF GRANULAR SOILS | STANDARD PENETRATION INDEX "N" VALUE (BLOWS/ft - 300mm) |
|---|---|
| Very loose | 0 - 4 |
| Loose | 4 - 10 |
| Compact | 10 - 30 |
| Dense | 30 - 50 |
| Very dense | > 50 |

| CONSISTANCY OF COHESIVE SOILS | UNDRAINED SHEAR STRENGTH (Cu) | |
|---|---|---|
| | (P.S.F.) | (kPa) |
| Very soft | < 250 | < 12 |
| Soft | 250 - 500 | 12 - 25 |
| Medium | 500 - 1000 | 25 - 50 |
| Stiff | 1000 - 2000 | 50 - 100 |
| Very stiff | 2000 - 4000 | 100 - 200 |
| Hard | > 4000 | > 200 |

**ROCK QUALITY DESIGNATION**

| "RQD" (%) VALUE | QUALIFICATIVE |
|---|---|
| < 25 | very poor |
| 25 - 50 | poor |
| 50 - 75 | fair |
| 75 - 90 | good |
| > 90 | excellent |

**STRATIGRAPHIC LEGEND**




## SAMPLES:

### TYPE AND NUMBER

The type of sample recovered is shown on the log by the abbreviation listed hereafter. The numbering of samples is sequential for each type of sample.

SS: Split spoon
SSE, GSE, AGE: Environnemental sampling
ST: Shelby tube
PS: Piston sample (Osterberg)
AG: Auger
RC: Rock core
GS: Grab sample

### RECOVERY

The recovery, shown as a percentage, is the ratio of length of the sample obtained to the distance the sampler was driven/pushed into the soil.

### RQD

The "Rock Quality Designation" or "RQD" value, expressed as a percentage, is the ratio of the total length of all core fragments of 4 inches (10cm) or more to the total length of the run.

## IN-SITU TESTS:

N: Standard penetration index
R: Refusal to penetration
$N_C$: Dynamic cone penetration index
Cu: Undrained shear strength
Pr: Pressuremeter
k: Permeability
ABS: Absorption (Packer test)

## LABORATORY TESTS:

$I_p$: Plasticity index
$W_l$: Liquid limit
$W_p$: Plastic limit
H: Hydrometer analysis
GSA: Grain size analysis
A: Atterberg limits
w: Water content
g: Unit weight
C: Consolidation
CS: Swedish fall cone
CHEM: Chemical analysis
O.V.: Organic vapor