GEOTECHNICAL SITE STUDY REPORT (PHASE 3)

VOLUME 1 OF 2

RABASKA LIMITED PARTNERSHIP

Rabaska – LNG Receiving Terminal
West Option Site
Levis, Quebec

Our File :
T-1050-C (604238)

May 2006

Terratech

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1. <u>INTRODUCTION</u>

The services of Terratech, a Division of SNC-Lavalin Environment Inc., were retained by Rabaska Limited Partnership to carry out a Phase 3 geotechnical site study at the proposed Rabaska - LNG Receiving Terminal, specifically at the selected West Option Site located in Levis, Quebec.

The reader is informed of the existence of four earlier geotechnical reports, namely:

- ***Terratech Report T-1050 (603794)*** issued on 30 July 2004 “Rabaska-LNG Receiving terminal, Levis / Beaumont, Quebec, Preliminary Geotechnical Consultation Report”. This document provided early and readily available information in terms of topography, hydrometry, bathymetry, geological soil and rock data, groundwater conditions and seismicity.

- ***Terratech Report T-1050-A (603333-RABA)*** issued on 10 March 2005 “Rabaska-LNG Receiving terminal, Levis / Beaumont, Quebec, Geotechnical Site Investigation Report (Phase 1)”. This document provided factual information from boreholes and seismic refraction surveys carried out during the fall of 2004, at four option sites initially considered for the project, together with the results of laboratory testing of soil and rock samples retrieved from the boreholes, including the boreholes of the W Series carried out at the West Option Site.

- ***Terratech Report T-1050-B (603333-KELL)*** issued on 4 May 2005 “Rabaska-LNG Receiving Terminal, West Option Site, Levis, Quebec, Geotechnical Site Investigation Report (Phase 2)”. This report is presented in two volumes: Volume 1 and 2. The document provided all factual information gathered at the selected West Option Site, and thus included the results of boreholes of the W Series (carried out in fall 2004) and boreholes of the BH-101-05 to BH-401-05 Series (put down during winter and spring 2005), and comprised seismic refraction surveys carried out during the fall of 2004, including down-hole seismicity data obtained in April 2005. The report also provided recommendations for the preliminary design of foundations of the proposed facilities.

- ***Terratech Report T-1050-B (603333-KELL) / Addendum to Volumes 1 and 2*** issued on 9 September 2005 “Rabaska-LNG Receiving Terminal, West Option Site, Levis, Quebec, Addendum to Volumes 1 and 2 / Geotechnical Site Investigation Report (Phase 2)”. This document provides additional information and recommendations based on readily available geotechnical and geological data, in response to comments from M. W. Kellogg Limited and the Industrial Division of SNC-Lavalin Inc.

This Phase 3 geotechnical study ***Report T-1050-C*** gathers all geological and geotechnical data previously obtained at the retained West Option Site, and thus includes all pertinent information previously provided in the aforementioned reports T-1050, T-1050-A, T-1050-B and T-1050-B (Addendum). This document also provided new input information from recently carried out (Fall 2005) boreholes, exploration trenches, trial rock excavations, additional down-hole seismic survey, electric soundings, and laboratory testing of soil and rock.

Subsurface investigations in the St. Lawrence River (at the site of the proposed Jetty) were not included in the scope of works of Terratech. These were performed by Laboratoires d’expertises de Québec Ltée (working for Roche Ltée Groupe-conseil) and by Procean Environnement inc. (SNC-Lavalin). At the specific request of Rabaska Limited Partnership, the results and technical reports relevant to these investigations were inserted in Appendix IX and Appendix X of Volume 2 of this report. A stratigraphic cross section of the entire project site, from the proposed docking facilities in the St. Lawrence River to the contemplated LNG Process Area is presented on the very last drawings inserted in Appendix III of this report (Volume 1).

Phase 3 subsurface exploration works at the West Option Site were carried out with the objective of specifically investigating the proposed LNG Process Area (and Lay-Down Area), Unloading Lines (Deep Rock Cut to the Jetty), and Access Roads and Paved Areas. The investigations were also aimed at determining rock features and anomalies by means of special trial rock excavations.

2. PROCEDURES

2.1 General information

Detailed procedures for Phases I through 3 of the geotechnical site study carried out at the selected West Option Site of the proposed Rabaska - LNG Receiving Terminal, are outlined in the following sections.

Subsurface investigations, soil description, and in-situ and laboratory testing of soil and rock were performed in compliance to the recognized standards listed in Appendix II. Standards applicable to chemical testing of groundwater are given at the end of Appendix II.

During the Phase 1 geotechnical site investigation, the West Option Site, was investigated by means of 7 boreholes, identified as W-001-04 to W-006-04, and W-008-04, totalling 104 m of drilling works. The boreholes were carried out during the period of 22 to 30 September 2004. Also, geophysical investigations involving 4 seismic refraction survey lines for a total of 3.4 km were carried out at and close to the West Option Site on 4 October 2004, from 18 to 21 October and from 11 to 13 November 2004.

During the Phase 2 geotechnical site investigation, the West Option Site was investigated by means of 23 boreholes, identified as BH-101-05 to BH-110-05, BH-111A-05, BH-116A-05, BH-116B-05, BH-117A-05, BH-117B-05, BH-301-05 to BH-307-05, and BH-401-05, totalling 637 m of drilling works. The site investigation also included 2 down-hole seismic surveys in open holes (BH-101-05 and BH-109-05) to determine soil and rock shear wave velocities and small strain dynamic properties. The Phase 2 subsurface investigations were carried out during the period of 8 February 2005 to 15 April 2005.

During the Phase 3 geotechnical site investigation, the West Option Site was investigated by means of 7 boreholes, identified as BH-501-05 to BH-507-05, totalling 121 m of drilling works. The site investigation also included 3 test pits, named TP-503-05 to TP-505-05, extending in the overburden to depths ranging from 1.5 to 1.7 m below existing grade, 2 trial excavations into rock (TE-A-05 and TE-B-05), and 2 vertical electric soundings (RT-1-05 and RT-2-05), and one down-hole seismic survey in an open hole (BH-501-05) to determine soil and rock shear wave velocities and small strain dynamic properties. The Phase 3 subsurface investigations were carried out during the period of 30 September to 4 November 2005.

The boreholes, exploration trenches, trial excavations, electric soundings and geophysical survey lines were located on site by Terratech personnel with respect to the SCOPQ-NAD83 system of coordinates. The elevation of the existing ground surface at the location of the field exploration works was determined in reference to the geodetic datum.

The location of the boreholes and geophysical lines is shown on Drawing T-1050-C-0000-4GDD-0001 included in Appendix III.

2.2 <u>Borehole Drilling and Sampling</u>

The boreholes of Phase 1 (W-Series) and of Phase 3 investigations (BH-501-05 to BH-507-05 Series) were carried out using a track mounted CME 55 rotary drill rig.

The boreholes of the Phase 2 investigation (BH-100-05 to BH-401-05 Series) were accomplished by means of two track mounted rotary drill rigs, although inclined boreholes (BH-116A-05 and BH-117A-05) were carried out with a rotary drill rig mounted on skids. The inclined boreholes BH-116A-05 and BH-117A-05 were put down with an inclination of about 50° from the horizontal. They were oriented towards the Northwest. During the course of the inclined holes, the inclination (from the horizon) was monitored by means of the Tropari Apparatus, which gave inclination and magnetic azimuth. The Tropari Apparatus, designated as Tropari/PDSI, was supplied by Pajari Instruments Ltd. (Ref.: Section 7.0).

The drill rigs were equipped with drill casing of sizes NW, HW and PW. The boreholes were terminated at depths ranging from 4.7 to 79.5 m below existing ground surface. Water was used as drilling fluid in all of the boreholes whose results are presented in Appendix I of this report.

Remolded soil samples were recovered from the boreholes using a standard 51 mm O.D. standard split-spoon sampler and a procedure that allowed the simultaneous determination of Standard Penetration Test N-values. A hammer of the “automatic and safety” type was used to drive the split-spoon sampler into the ground. Clayey or cohesive and stiff to very stiff soils were encountered only sparsely, very locally and within thin layers at the investigated site. Therefore, only disturbed soil samples were taken in the boreholes by means on the standard split spoon sampler.

The soil sampling by means of the standard split-spoon sampler (and SPT testing) was generally carried out at depth intervals of 0.8 m. However locally in Boreholes BH-401-04, BH-501-05, BH-503-05 and W-004-04, soil samples were taken at 0.8 m intervals down to depths ranging from 6.0 to 10.7 m below existing ground surface, and at intervals of 1.5 m thereafter. No soil sample was retrieved from the inclined boreholes (BH-116A-05 and BH-117A-05). In some boreholes, core drilling techniques were used to sample and traverse dense to very dense soils containing cobbles and boulders.

All recovered soils samples were visually examined in the field by Terratech senior soil technicians. They were placed in plastic bags and sent to Terratech Laboratory in Montreal for laboratory testing and storage. Visual description of soils was done in compliance to the classification and terminology provided in Appendix I (see: Explanation of the Form Boring Log). Applicable test standards are listed in Appendix II of this report.

Bedrock was encountered and core drilled in all boreholes, except in Boreholes BH-401-05, BH-504-05 and BH-506-05 which were terminated in the overburden. In most of the boreholes, the bedrock was drilled in NQ-3 size core barrel. Locally (Boreholes W-004-04, BH-102-05, BH-104-05, BH-106-05, BH-110-05, and BH-111A-05), HQ or HQ3 core barrels were used for rock core drilling. In Boreholes BH-101-05, BH-109-05 and BH-501-05, the bedrock was drilled in PQ size core barrel, and the said boreholes were provided with 63.5 mm internal diameter and bottom capped PVC lining grouted in place with a ciment-bentonite mixture, to allow down-hole seismicity tests to be performed.

After completion of the boreholes, bottom perforated plastic standpipes were inserted in the boreholes drilled in NW / NQ3 size, to allow groundwater level observations. In Boreholes drilled in HW / HQ or HQ3 size (W-004-04, BH-102-05, BH-104-05, BH-106-05, BH-110-05, and BH-111A-05), 50 mm diameter PVC tubes were inserted in the completed holes. Each PVC tube was provided with a slotted bottom portion 6.1 m in length. A peripheral sand jacket was placed at the outset of the slotted portion of the tube, and bentonite seal was provided above the slotted portion, as to convert the borehole into an observation well (or piezometer) for groundwater sampling and monitoring. Borehole BH-503-05 was equipped with a 20 mm size open end Casagrande type piezometer tube. Schematic information on the main components of the above groundwater observation wells and piezometer is shown on the Boring Logs included in Appendix I.

The detailed description of the various soil layers and bedrock encountered in the boreholes are presented on the boring logs included in Appendix I.

The soil and rock samples retrieved from boreholes and exploration trenches during Phases 1 to 3, that were not used for testing purposes, will be stored at Terratech Laboratory in Montreal until 31 December 2008, which is considered as a practical foreseeable future. At that time, Rabaska Limited Partnership or its representatives shall be consulted about future use or disposal of the said samples.

2.3 Exploration Trenches / Test Pits

Three shallow exploration trenches or test pits, numbered TP-503-05 through TP-505-05, were put down in the overburden to depths ranging from 1.5 to 1.7 m below existing grade. They were carried out on 14 October 2005, by means of a backhoe (Caterpillar 430), at the site of a potential lay-down area. The purpose of this investigation was to retrieve large size soil samples mainly for Proctor compaction and CBR testing. The test pits were put down respectively in the immediate vicinity of Boreholes BH-503-05 through BH-505-05.

The detailed description of the various soil layers encountered in the test pits are presented on test pit logs inserted at the very end of Appendix I (after the boring logs).

2.4 Trial Excavations

Two trial excavations, identified TE-A-05 and TE-B-05, were carried out at the project site during the period of 12 to 21 October 2005, for the main purpose of assessing the rock cartography and the bedrock structure. The location of the trial excavations is shown on Drawing T-1050-C-0000-4GDD-0001 in Appendix III.

Trial Excavation TE-A-05 is situated at the crest of the rock plateau some 200 m south of the St. Lawrence shoreline and is oriented in a SE to NW direction (N 135º / N 315º). The position of this trial excavation corresponds to the south limit of a proposed Deep Rock Cut leading to a future Harbour Facility or Jetty. The trench has a width of about 3 m, a length of 10.0 m and an average depth 2.5 m below existing ground surface. It was extended in depth to about 2.0 to 2.3 m into the bedrock. The excavation was performed by means of a Caterpillar 225 LC Excavator using a 1 m^3 size bucket.

Trial Excavation TE-B-05 is located near the proposed West Storage Tank, at the site of an inferred rock anomaly, and is oriented in SE to NW direction (N 135º / N 315º). The location of this trial excavation was selected on the basis of site accessibility for the excavator (to limit damage to the property), further to be within the alignment of rock anomalies previously assessed by the geophysical survey lines. The excavation has a bottom width of about 2.5 to 3.0 m and a length of 45 m. With an average depth of 4 m below existing ground surface, the excavation was extended approximately 0.6 to 3.0 m into the overburden and some 0.5 to 2.0 m into bedrock. The excavation was performed by means of a Caterpillar 235 Excavator equipped with a 1.7 m^3 size bucket.

The main objectives of the trial excavations were to investigate, within the upper strata of the bedrock, specific and detailed rock features, such as folds, dips, anomalies, closely jointed stratigraphy, rock quality. The excavations were also used to assess the performance of standard excavators into the shallow layers of bedrock.

The observed overburden and bedrock features at the trial excavations are shown on Drawings T-1050-C-0000-4GDD-0005 and T-1050-C-0000-4GDD-0006 in Appendix III.

2.5 Geophysical Investigations

Geophysical investigations were carried out by Geophysics GPR International Inc. at the West Option Site. These included 4 seismic refraction survey lines, 3 down-hole seismic surveys in Boreholes BH-101-05, BH-109-05 and BH-501-05, and 2 vertical electric soundings.

2.5.1 Seismic Refraction Surveys

The seismic refraction survey lines varied in length from 600 to 1200 m, for a total of 3.4 km. They are located within or close to the West Option Site, at the following locations:

- Beyond southeast sector: Line GW-002-04;
- Northeast limit: Line GN-001-04;
- Northeast sector: Line GN-001A-04;
- Southwest sector: Line GN-001B-04.

The purpose of the seismic refraction surveys was to produce depth profiles for layers of overburden and for bedrock, and also to identify zones of alteration or weaknesses within the bedrock, in order to verify or/and locate possible fault zones, as expected from available regional geological maps.

Geophysical seismic refraction lines GW-002-04 and GN-001-04 were carried out on 4 October and during the period of 18 to 21 October 2004. Geophysical Lines GN-001A-04 and GN-001B-04 were accomplished from 11 to 13 November 2004. The conventional seismic refraction method, which is in reference to the usual practice for soils and bedrock surface, was applied to all four survey lines. For details, the reader should refer to Appendix VII (section 4.1 of the report by Geophysics GPR International Inc., dated 2 March 2005). However, with survey lines GN-001A-04 and GN-001B-04, special on-site testing and interpretation designated as seismic resonance or "TISAR" were also performed. The term "TISAR" is an acronym for Testing & Imaging using Seismic Acoustic Resonance. Procedures related to "TISAR" interpretation are discussed in Appendix VII (section 4.2 of the report by Geophysics GPR International Inc., dated 2 March 2005).

Detailed procedures, results and limitations relevant to the seismic refraction surveys are presented in Appendix VII, in Report M-04958 issued in March 2005 by Geophysics GPR International Inc.

2.5.2 Down-Hole Seismic Surveys

Down-hole seismic tests were performed by Geophysics GPR International Inc. in Boreholes BH-101-05 and BH-109-05, to the maximum borehole depth of 25 m, and also in Borehole BH-501-05 to a depth of 19 m. The objective of the down-hole surveys was to determine soil and bedrock shear wave velocities as well as low strain dynamic parameters. The surveys were carried out on 3 April 2005 (BH-101-05 and BH-109-05) and on 3 and 4 November 2005 (BH -501-05). There was a 0.6 to 1.0 month difference between the installation of the casing (with peripheral bentonite-cement grouting) and the doing the down-hole tests. This is believed to have a negligible impact on the test results from delamination of the grout.

Detailed procedures, results and limitations relevant to the down-hole seismic surveys are presented in Appendix VIII, in Report M-05043 issued on 22 April 2005 an in Report M-05128 issued on 29 November 2005 by Geophysics GPR International Inc.

2.5.3 <u>Vertical Electric Soundings</u>

Two vertical electric soundings, identified TR-1-05 and TR-2-05, were carried out at the site of the proposed LNG Process Plant. The soundings were performed on 3 and 4 November 2005 by Geophysics G.P.R. International inc.

Detailed procedures, results and limitations relevant to the vertical electric soundings are presented in Appendix VIII, in Report M-05128 issued on 29 November 2005 by Geophysics GPR International Inc.

2.6 <u>Laboratory Testing</u>

2.6.1 Soil Testing

Laboratory testing of soil and rock were performed in compliance to the recognized standards listed in Appendix II of this report. Standards applicable to chemical testing of groundwater are given at the end of Appendix II.

On selected and representative soil samples recovered from the boreholes, grain size analyses, and moisture content determinations were carried out in the laboratory of Terratech, to complement the visual soils descriptions. Moisture content and Atterberg limits determinations were also done on clay soil samples recovered from the boreholes. The results of the moisture content and Atterberg limit determinations on soil samples are shown on borehole logs in Appendix I. Grain size curves and tabulated results of moisture content and Atterberg limits are presented in Appendix II.

On representative soil samples retrieved from the Test Pits TP-503-05 to TP-505-05, the following set of laboratory testing was performed: natural moisture content, Modified Proctor compaction test, and California Bearing Ratio (CBR) determinations following a 96 hour soaking period. The detailed results of this testing are presented in Appendix II.

2.6.2 Rock Testing

All rock cores retrieved from the boreholes were visually examined on site by the senior geotechnical technician supervising the drilling. Later in the laboratory, the recovered rock cores were submitted to a detailed structural description performed by Terratech licensed geologists.

Also on selected and rather intact or suitable rock cores originating from the West Option Site, unit weight and uniaxial compressive strength determinations were carried out in the laboratory.

Unit weights were measured on rock cores retrieved from Boreholes of the W-002-04 to W-008-04 Series and subsequently submitted to compressive strength determinations. In boreholes of the BH-102-05 to BH-110-05 Series and of the W-002-04 to W-008-04 Series, the rock compressive strength determinations were not performed in true compliance to the applicable standards (ASTM D 2938 "Standard Test Method for Unconfined Compressive Strength of Intact Rock Core Specimens"). The tests were carried out on an hydraulic press normally used to perform compression tests on concrete cylinders. In general, the tested rock cores were grinded and capped with high resistance sulfur compound before being submitted to the compression tests. This procedure, which somewhat deviates from the standard test protocol, is inferred to yield test results that would be on the low margin of standard test results. With due consideration to the generally very poor to poor quality of the rock, which allowed only the best segments of rock cores to be submitted to the testing, this deviation from the standard test procedure is considered acceptable.

On Boreholes BH-505-05 and BH-507-05, uniaxial unconfined compressive strength determinations were carried out as per ASTM Standard D 2938 on rather best quality rock cores. The tested cock cores are inferred to belong to the good or excellent quality bedrock, as rock cores originating from very poor to fair quality rock could not be tested for compressive strength.

Pyrite detection tests in compliance to the Quebec Standard NQ 2560-500 Procedure were undertaken on 3 rock cores sampled at shallow depth in boreholes of the W Series, and also on 18 rock core segments retrieved from boreholes of the BH-101-05 to BH-401-05 Series at depth ranging from 7 to 13 m below ground surface. The purpose of the testing was to assess the rock swelling potential due to the presence of pyrite. Results of the pyrite detection tests are presented in Appendix II.

Photographs were taken of all rock cores. On the rock core photographs of boreholes of the W-Series, depths are given in feet. The reader should therefore refer to the tables of the structural description of bedrock (Appendix V) to obtain a direct depth conversion in meters. Photographs of the rock cores of the boreholes of the BH-Series are given in meters. Photographs of all rock cores are included in Appendix IV.

The detailed structural description of the bedrock is presented in Appendix V.

2.6.3 Groundwater Testing

Groundwater samples were taken on 14 April 2005 from Boreholes BH-102-05, BH-104-05, BH-106-05 and BH-110-05. The said boreholes, which were converted in observation wells, were purged prior to the groundwater sampling. The following chemical analyses were performed on groundwater by Maxxam Analytics inc.:

- pH
- Sulfur anion (S=)
- Alkalinity (Total as CaCO3) pH 4.5
- Bicarbonates (HCO_3 as $CaCO_3$)
- Carbonate (CO_3 as $CaCO_3$)
- Chloride (Cl)
- Sulfates (SO_4)

The results of the testing are presented and discussed in Section 5 of this report. Analytical Report pertaining to the chemical testing of groundwater are inserted at end of Appendix II.

3. GEOLOGY AND SEISMICITY

In reference to published soil and bedrock geological and seismic information, as outlined in Section 7 of this report, and with due consideration to soils and bedrock features recently observed at the site, the following sections are provided.

3.1 Soil Deposits

In LaSalle 1978, soils or overburden on the project site were designated as high terrace well sorted marine sand. Based on that publication, the overburden at the contemplated West Option Site was inferred to vary from less than 3 m in thickness, although to the west the soils were believed to exceed 3 m. Locally within the northeastern part of the project site, a poorly drained marsh area was inferred to contain some peat, with bedrock at shallow depth. At the site of the Hydro-Quebec power line and within about 0.5 km north of the said line, investigations carried out in 1963 by Terratech indicated the presence of compact to dense silty sand with gravel, with bedrock at depths of about 1 to 2 m, or locally at 5 m.

Investigations, recently carried out within the project site and adjacent area, generally indicated, under 0.1 to 0.3 m of topsoil (and locally under 0.5 to 0.9 m of surface peat), the presence of compact sand with silt and gravel, extending to about 1 to 6 m below existing grade. These are locally underlain by a layer of stiff to very stiff clay soils some 0.4 to 1.7 m in thickness and extending no deeper than 0.9 to 4.9 m below grade. Dense to very dense glacial till consisting of sand with some silt and gravel and occasional cobbles and boulders, is encountered beyond 1.2 to 6.1 m depth, generally extending down to depths of the order of 2.7 to 13.2 m (and locally to 24 m), where bedrock was encountered.

A drawing showing the elevation of bedrock is presented in Appendix I. Based on this drawing, two rock depressions were found some 150 m southwest and 270 m south of the proposed West Storage Tank, i.e. near Borehole BH-503-05 and Seismic Line GN-001B-04 (at metric point 0+729).

A poorly drained and peat covered (marsh) zone is present within the eastern sector of the project site, expanding some 250 m northeast of the proposed East Storage Tank.

3.2 Bedrock

3.2.1 Regional geological context

In the region of the project area, the sedimentary rocks overlying the Precambrian crystalline basement belong to the Appalachian Geological Province and are of lower Paleozoïc age. Based on Saint-Julien 1995, these rocks form a lithotectonic domain called the Bacchus Nappe. This structural domain, is limited in the northwest by the overthrust Logan Fault which runs through the St. Lawrence River and marks the front on the Appalachian Mountain Belt.

The Bacchus Nappe is in faulted contact with the adjacent nappes and overlies younger rock formations. For example, its Cambrian base overlies the Lower and Middle Ordovician terranes west of the site (Levy Nappe and Quebec Nappe).

Generally, the geology on the south shore of the St. Lawrence River in front of Quebec City consists mainly of thinly folded and faulted strata, imbricated and piled together along large, deep and reversed fault planes, gently dipping with depth, toward the southeast. During the taconian orogeny, the sediments were pushed over the continental platform approximately from the southeast to the northwest, thus forming stacking lithostratigraphic units piles called nappes. This mechanism could be compared, at a very large scale, to a sliding card deck.

In the vicinity (5 km SW) of the project site, the total thickness of the piled nappes covering the Grenville sub-basement is estimated at 4 km approximately. The Bacchus Nappe lies on top of the sequence and its stratigraphic sequence reaches a thickness of approximately 1 km as mentioned in the available governmental geological documents. The reader is invited to review some of the references listed in Section 7 of this report.

3.2.2 Local geology

The Bacchus Nappe, which outcrops locally within a short range of the project site, is composed, from base to summit, of the following three rock formations (St-Julien, 1995):

- The L'Anse Maranda Formation;
- The Lauzon Formation;

- The Pointe de la Martinière Formation.

The above Lower Cambrian to Lower Ordovician lithostratigraphic units are mainly composed of shales and multicolor mudstones interbedded with siltstones, sandstones and calcareous conglomerates. Except locally, the contacts between the three formations do not show angular discordance.

The main body of the Bacchus nappe is characterized by the presence of numerous imbricated overthrust faults which repeat the stratigraphy inside the nappe. They are oriented NNE-SSW to N-S on the south shore, and dip toward east and southeast. In addition, the formations are folded (asymmetric folds).

At the project site, the underlying bedrock belongs to the Pointe de la Martinière Formation. It generally consists of thinly bedded (20 to 300 mm thick) red and green shales and mudstones, dolomitic mudstones, black micaceous shales, grey shales, siltstones, some limestones (less than 300 mm thick) interbedded with grey shales and finally, calcareous conglomerates. The total thickness of the formation is estimated at 350 m (St-Julien 1995).

The beddings are generally oriented NNE-SSW, and dip to the east. This pattern is often disturbed by the presence of folds which locally form series of anticlines (or antiforms) and synclines (or synforms) generally plunging 10 to 20° to the south.

The mudstones are usually massive, whereas the shales remain more fissile. The mudstone and shale locally present a slaty cleavage parallel to the axial planes of folds.

Investigations carried out at the project site by means of seismic refraction geophysical surveys and diamond core drilling have provided valuable information concerning the quality of the rock and in some extent its structure. Subsurface investigations carried out at the project site by boreholes, have also revealed the nature and properties of soils and the true position of bedrock. Two trial excavations were carried out to assess the rock cartography and also the bedrock structure with respect to rock anomalies, folds, faults, and synforms.

The detailed rock core description are presented in Appendix V, whereas the photographs of rock cores are inserted in Appendix IV. The seismic refraction geophysical report is included in Appendix VII.

The main geological features of the rock are outlined and discussed in the following sections.

3.2.2.1 Geophysical surveys

From the seismic refraction geophysical surveys, the bedrock profile and the rock seismic velocities were determined. Based on the results of the surveys, the rock quality was found to vary substantially, thus ranging from anomalous to sound rock. The heterogeneity noticed in the bedrock velocities seems to reflect the lithological pattern observed on site.

Some anomalous targets were specifically investigated by means of diamond core drilling. In these cases (see Boreholes W-003-04 and W-005-04), the core recovery indicated a poor quality rock with very low RQD values, especially near the bedrock surface. This finding is compatible with the geophysical results.

The seismic resonance survey highlighted some planar features, which may be interpreted in some cases as shear zones or fault zones. The seismic resonance also showed some bedding trends and some folding features such as the synform shape interpreted on Seismic Line GN-001A-04. In all, the geophysical surveys reflect the general structure of the bedrock.

Trial excavations were carried out (see Section 3.2.2.3) to assess the bedrock structure with respect to rock anomalies, folds, faults, and synforms.

3.2.2.2 Rock core drilling

From the close examination of the rock cores retrieved from the boreholes, the rock facies intersected at the site were found to be similar to those described in the literature (ref.: Appendix I and Section 4.3 of this report).

Based on RQD values, the rock quality may be described as very poor to fair in general, and occasionally ranging from good to excellent. It is worth to remind the reader that, in this type of rock, especially in laminated or thinly layered sections such as shales, where parting along the bedding planes occurs easily, the RQD evaluation is influenced by the number of lithologic joints and their features, i.e. whether they are induced or natural. The judgment of the geologist is therefore required. For this reason, RQD values are often underevaluated.

In this study, the RQD values do not reflect completely and thoroughly the rock quality. Therefore, the RQD should be used as a guide to discriminate the relative rock quality over sections within a same borehole.

Few targets identified by means of the geophysical surveys, were investigated particularly to verify the presence of faults, or inferred faults. Also boreholes were performed at the project site to determine the quality of the bedrock.

Two boreholes, one inclined towards the northwest and one vertical hole (Boreholes BH-117A-05 and BH-117B-05) were located along Seismic Refraction Line GN-001-04 close to borehole W-003-04. At this location, the rock in the inclined hole (BH-117A-05) shows evidences of folding near 16.2 m depth, and at a deeper depth the rock is fragmented and silty. Microfolds and minor movements in the sediments are also reported. At a deeper depth in the hole, calcite veins and hairy veinlets are present.

In the vertical borehole (Borehole BH-117B-05), the lithology is similar down to 28.5 m below existing grade, whereas it is followed by a red mudstone which is not intercepted in the adjacent incline hole.

These holes were drilled close to a narrow fold hinge as shown on the St-Julien map. Based on scarce evidences, the presence of a fault is not clearly determined. However, in the area, as shown on sections by St-Julien, the narrow folds are usually faulted, and this could therefore be the case here. Meanwhile, the predominant structures encountered in the holes are believed to be in relation with the folding. Nevertheless, the presence of a faulted fold remains highly possible. Additional information concerning past fault activity is provided at end of Section 3.2.2.3.

In borehole BH-108-05, between depths of 52 m and 64 m approx., the rock is anomalous, of poor quality and probably faulted. Calcite veinlets and veins, frequent slickensides and fault striations on joint surfaces are described. At a depth of 62 m, a probable fault breccia (cemented with calcite) is reported. The nature of this perturbed rock section is not fully understood. However, on the photographs, the fault breccia appears to be healed.

3.2.2.3 Trial Excavations

Two trial excavations, identified TE-A-05 and TE-B-05, were carried out at the project site for the main purposes of assessing the rock cartography and the bedrock structure. The location of the trial excavations is shown on Drawing T-1050-C-0000-4GDD-00001 in Appendix III.

The following sections depict soil and bedrock conditions and features, as observed in the trial excavations.

- ***Trial Excavation TE-A-05***

Trial Excavation TE-A-05 is situated at the crest of the rock plateau some 200 m south of the St. Lawrence shoreline. The detailed results Trial Excavation TE-A-05 are shown on Drawing no T-1050-C-0000-4GDD-0005 in Appendix III.

This excavation was about 3 m in width and 10 m in length. It was extended, by means of a Caterpillar 225 LC Excavator using a 1 m^3 size bucket, to an average depth 2.5 m below existing ground surface, and to depths of the order of 2.0 to 2.3 m into the bedrock. This equipment was able to easily complete the excavation within a 2.5 hour period.

In this excavation, the overburden averages a thickness of 0.2 to 0.5 m. It is exclusively composed of brown reddish sand and silt with some gravel, with also some roots close to the natural ground surface.

At the exposed bottom of the excavation, the bedrock is fragmented and slightly weathered within its first 0.3 m, whereas at greater depth it becomes of relative good quality. The bedrock consists of a succession of green and red mudstones beds (some 10 to 300 mm thick), mostly slaty with traces of dark shale interbeds (10 mm thick). Along the entire length of the excavation, rock beddings are typically oriented N 30º, with dips ranging from 58º to 64º. Numerous joints were observed, belonging to three main families (see Drawing no T-1050-C-0000-4GDD-0005, in Appendix III).

Very minor water inflows were observed at the bottom of the excavation, originating mainly from the bedrock. These were easily controlled and evacuated by pumping .

- ***Trial Excavation TE-B-05***

Trial Excavation TE-B-05 is located near the proposed West Storage Tank at the site of an inferred rock anomaly. The location of this trial excavation was selected with the deliberate intention of intercepting the potential alignment of rock anomalies previously assessed by the geophysical survey lines. Positioning the trial excavation was also done on the basis of practical site accessibility for the excavator, mainly to limit damage to the property and with due consideration of access limitations to the project site. The reader is reminded that the “possible faults” shown on the appended Drawing T-1050-C-0000-4GDD-0001 (Appendix III) were directly transcripted from the geological maps. These “possible fault” lines, which are located at least some 0.2 to 0.5 km from the contemplated LNG facilities, have provided no specific or clear signs of rock anomalies during the geophysical surveys. However, signs of rock anomalies were locally disclosed elsewhere along the geophysical survey lines. In view of this, it has become desirable to have a direct look at the rock anomalies by stretching out Trial Excavation TE-B-05 some 20 to 25 m on each side of the alignment of anomalous rock features previously assessed by the geophysical survey lines. The detailed results of Trial Excavation TE-B-05 are shown on Drawing no T-1050-C-0000-4GDD-0006, in Appendix III.

The excavation has a bottom width close to 2.5 or 3.0 m and a length of 45 m. The excavation was performed by means of a Caterpillar 235 Excavator using a 1.7 m^3 size bucket, and was extended to an average depth of 4 m below the existing ground surface. The excavation generally comprises some 0.6 to 3.0 m of overburden, plus 0.5 to 2.0 m of bedrock, as these depths were deemed sufficient to observe shallow rock anomalies and possible movement or disturbance in the soils that could be related fault activity.

During the field work related to this trial excavation, which lasted about 4 days, water inflows due the high ground water condition concurred to flood the excavation as limited on-site pumping equipment was then available. In spite of this, it was estimated that the excavator would have been able to complete the excavation (soil and rock) within about a 5 hour (half day) period with adequate pumping equipment. Pumping of the water, to lower the groundwater table down to the exposed bottom of the excavation, was achieved by means of a high capacity pump. To completely dry-out the flooded excavation with a high capacity pump took about 2 hours. During the rather short time period needed to draw-down of the water table, limited silt and sand sloughings were observed on the exposed 1.5 (H) : 1.0 (V) sand and silt and clayey slopes of the excavation.

Numerous water inflows were observed in the open excavation, often originating from the bedrock and generally associated with fissile rock partings. The water inflows were continuous but their intensity decreased after two days (upon uninterrupted pumping).

In this trial excavation, the depth of the overburden typically reaches 3 m, consisting of variable thicknesses of sand, silt and gravel, atop of a 1 m thick layer of grey clay itself overlaying a rather continuous and very stiff reddish horizon of silt some 0.5 m in thickness. This basal deposit is underlain by bedrock.

As observed on the total length of the excavation and specifically along its western side, the bedrock surface is irregular and undulating. This, in some extent, highlights the presence of folds hinges and steep rock beddings. The bedrock is usually fragmented and slightly to moderately weathered from the surface to a depth of 1.5 m where it becomes sounder. In some areas of the trench, the exposed rock was also found in very fragmented, softer and highly weathered conditions, at least within 1 m depth.

The observed bedrock consists essentially of alternating greenish grey and pale to dark grey mudstones, sometimes sandy and slaty, and interbedded with generally thin layers of dark grey to black shales. The shales occur in variable proportions (see the legend on Drawing T-1050-C-0000-4GDD-0006). Occasionally, thin (approximately 30 mm) calcareous and siltstone or fine grained sandstone horizons are present. Some sparse calcite veinlets are also observed.

The rock formation is typically folded with trends roughly northeast / southwest (N 035º / N 215º), in reference to True North. Along the trench, successions of minor synforms and antiforms were noticed. Evidences of movement such as striations (almost parallel to the dip direction of the beddings) and polished rock faces, are often visible over softer rock facies bedding surfaces, especially in the very fine grain rock and in the black shale. These movement features appear to be linked to the general folding process.

One of the main objectives for digging Trial Excavation TE-B-05 was to locate within a short distance from the Storage Tanks, and also whenever possible to investigate, the rock anomaly previously assessed from the geophysical survey. Based on the results of the survey, the projection of the anomaly along a northeast - southwest line is expected to cross the trench alignment at a distance of 30 measured from the southeast end of the trial excavation. However, the position of the anomaly remains then somewhat approximate, although probable.

In Trial Excavation TE-B-05, at distances ranging from 30 to 35 m from the southeast end of the trench, the bedrock is sheared and consists of weathered and well fragmented greenish mudstone and schitose black shale. This feature is visible, in an equal proportion especially on the west side of the trench. Some “hairy calcite veinlets” without definite pattern and few closely spaced joints steeply dipping to the north are also present. This poor rock quality formation is stacked between more competent formations.

The above features could indicate a rock anomaly related to a limited shear zone associated with foldings and parallel to the bedding planes. Similar features were also observed in Boreholes BH-507-05, BH-103-05 and BH-104-05, and in Borehole BH-108-05 which presents evidences of minor faulting such as secondary calcite fillings. The rock appeared to be healed.

Trial Excavation TE-B-05 was carried out in the glacial deposit to observe rock features and any movement or disturbance in the soils that could be related to a fault activity since the drawback of the glacier (-12 to -9.5 ky), whereas the aforementioned rock foldings and shear zones are believed to be related to the Appalachian Front (-450 My to –400 My). In this respect, while this excavation was being carried out, attention paid to the overburden has provided no clear evidence that the soil materials were disturbed to the bedrock otherwise that by human activity.

3.3 Seismicity

A site specific hazard assessment study is presently underway, and shall be inserted in a Seismic Hazard Report to be issued in a near future. This incoming report is intended to cover items such as earthquake history, local faults and fault activity, seismic hazard, and soil liquefaction potential.

4. SOIL AND BEDROCK DESCRIPTION

4.1 General consideration

The description of the various soil layers and of the bedrock encountered in the boreholes are presented in the Boring Logs included in Appendix I. The bedrock contour elevation are shown on Drawing # T-1050-C-0000-4GDD-0001 inserted in Appendix I.

Tabulated soil N_{SPT} values, and also rock RQD and compressive strength values are shown on the boreholes logs with respect to the depth and elevation. This information is presented in Appendix 1.

It should be noted that the subject site investigation was carried out for geotechnical purposes only. Thus, an environmental characterization of the site was beyond the scope of this mandate and as such, the soil descriptions provided herein shall not be used to ascertain the presence, or absence of contamination.

Soil and bedrock conditions are summarized in the following sections specifically for the contemplated West Option Site.

4.2 Soils

A summary of the soil conditions at the site is presented below.

4.2.1 Topsoil or Peat

Topsoil varying in thickness from 0.10 to 0.30 m was generally encountered in the vicinity of the West Storage Tank (Boreholes BH-101-05 to BH-105-05, BH-116B-05), at the site of the proposed Unloading Lines (Boreholes BH-301-05 to BH-307-05, and BH-507-05) and also at the location and in the near vicinity of the LNG Process Area (Boreholes BH-401-05, BH-501-05 to BH-506-05, W-001-04, W-002-04 and W-006-04, and test Pits TP-503-05 to TP-505-05). Locally (Boreholes BH-303-05 and BH-305-05 and W-004-04), no topsoil but only fill materials were found at ground surface.

At the site of the East Storage Tank, peat locally combined with top soils or overlain by fill materials was encountered, extending to depths of about 0.5 to 0.9 m below existing ground surface (Boreholes BH-106-05 to BH-111A-05). North and east of the East Storage Tank area (Boreholes W-003-04, BH-117B-05 and W-005-04), peat, covered by 0.3 to 0.6 m of fill, was found to extend to depths in the range 1.2 to 2.1 m below existing grade. The peat is generally fibrous at shallow depth and becomes amorphous with depth.

4.2.2 Generally Compact Sand with silt and gravel

At the site of the West Storage Tank (BH-101-05 to BH-105-05) and in its vicinity (W-002-04 and W-004-04), compact sand and gravel with some silt was identified below the topsoil (or locally peat) down to depths ranging from 1.4 to 2.3 m.

This same soil, or locally sand with some or trace of silt and gravel, was also found, in a compact to locally loose state of relative density, under the topsoil or fill covered peat at the site of the East Storage Tank and some distance north, east and west thereof (BH-106-05 to BH-111A-05, BH-116B-05, BH-117B-05, W-003-04 and W-005-04), extending to depths in the range of 1.5 to 5.0 m below grade.

Within the proposed LNG Process Area (BH-401-05, BH-501-05 to BH-506-05, W-002-04, W-004-04, W-006-04), compact sand with silt and gravel was generally encountered, extending to depths ranging from 0.8 to 6.1 m below existing ground surface.

Generally along the proposed unloading lines (BH-301-05 to BH-307-05, and BH-507-05), loose to compact sand with some silt and gravel was identified down to depths of the order of 0.6 to 3.1 m below grade, and locally to bedrock (BH-303-05, BH-305-05 to BH-307-05, and BH-507-05).

Results of laboratory testing obtained on this stratum are be summarized in Table 4-1.

Table 4-1
Results of Laboratory Testing
Compact Sand with Silt and Gravel

Tests	Number of tests	Unit	Results		
			Minimum	Maximum	Average
Grain size analyses (gravel, 5 to 80 mm)	13	%	4.0	54.9	24.4
Grain size analyses (sand, 0.08 to 5 mm)	13	%	21.0	65.0	45.4
Grain size analyses (fines, < 0.080 mm)	13	%	8.0	59.2	30.1
Grain size analyses (clay size, < 0.002 mm)	6	%	4.8	18.8	9.8
Moisture content	12	%	9.6	14.5	11.7
Modified Proctor (Maximum dry unit weight)	3	kN/m^3	20.4	21.5	20.8
Modified Proctor (Optimum moisture content)	3	%	7.8	8.5	8.2
California Bearing Ratio following a 96 hour soaking period (at moisture content varying from 0.8 to 4.5 % above the optimum moisture, and within a degree of compaction of 92 to 98 % of the Modified Proctor maximum dry density)	9	%	1	22	13

4.2.3 Firm to Very Stiff Clayey Soils

Clayey soils were encountered locally (BH-101-05, BH-103-05, BH-104-05, BH-106-05 to BH-108-05, BH-111A-05, BH-501-05, BH-502-05, and W-003-04), generally at depths of the order of 1.5 to 2.7 m, and extending to depths ranging from 1.8 to 5.2 m below existing ground surface. They ranged from stiff to very stiff sandy and clayey silt, to firm to stiff clay with some silt and gravel.

Very locally at the site of the Unloading Lines (BH-301-05), stiff to very stiff clay was identified below topsoil and down to 0.9 m below existing grade, where bedrock was encountered.

Results of laboratory testing obtained on this stratum are be summarized in Table 4-2.

Table 4-2
Results of Laboratory Testing
Firm to Very Stiff Clayey Soils

Tests	Number of tests	Unit	Results		
			Minimum	Maximum	Average
Grain size analyses (gravel, 5 to 80 mm)	6	%	0	12	4
Grain size analyses (sand, 0.08 to 5 mm)	6	%	3	52	20
Grain size analyses (fines, < 0.080 mm)	6	%	36	97	76
Grain size analyses (clay size, < 0.002 mm)	6	%	12	37	26
Natural moisture content	10	%	10	24	18
Limit of plasticity	5	%	13	19	17
Limit of liquidity	5	%	25	34	30
Plasticity Index	5	%	10	15	13

4.2.4 Dense to Very Dense and Well Graded Gravely Sand with Some Silt

At the site of the West Storage Tank (BH-101-05 to BH-105-05), dense to very dense soils were identified at depths of the order of 1.7 to 4.2 m, varying in gradation from gravely silt and sand, to sand with some gravel and silt, or sand and silt. They were found to extend to depths 7.7 to 11.1 m, where bedrock was encountered. Close to West Tank (W-004-04), dense to very dense sand with some silt and gravel, and silt and sand with trace of gravel and clay and occasional cobbles and boulders were found between depths of 1.8 and 11.3 m, underlain by bedrock.

Within the footprint of the East Storage Tank and its neighbouring area (BH-106-05 to BH-111A-05, BH-116B-05, BH-117B-05 and W-003-04) dense to very dense gravely sand with some silt and occasional cobbles, and silty and gravely sand were encountered at depths ranging from 3.4 to 5.0 m below ground surface. They were found to extend to depths of the order of 5.1 to 7.3 m (BH-106-05 to BH-110-05), and to depths of about 5.6 to 12.5 m below grade (BH-111A-05, BH-116B-05, BH-117B-05 and W-003-04).

Very dense sand with silt and gravel was found locally at the site of the Unloading Lines (BH-302-05 and BH-304-05) at depths of 3.1 and 2.1 m, and down to 4.6 and 2.7 m where bedrock was met.

Within the proposed LNG Process Area and the adjacent area (BH-401-05, BH-501-05, BH-503-05, BH-504-05, BH-506-05 and W-004-04), dense soils were found at depths of the order of 1.2 to 6.1 below existing grade, consisting of sand with some silt and gravel, and occasional cobbles and boulders, and locally sand and silt with trace and occasional gravel. In Boreholes BH-501-05, BH-503-05, W-004-04, dense and very dense soils reached bedrock at depths of the order of 11.3 to 24.0 m below grade. In Borehole BH-401-04, dense silt and gravel was encountered between depths of 4.6 to 10.7 m, followed by silty sand with occasional gravel presumably to about 17.5 m below grade and underlain by dense soils extending at least to 20.1 m depth where dynamic penetration tests were terminated on refusal, without formal rock determination. Boreholes BH-504-05 and BH-506-05 were also terminated within the dense to very dense soils respectively at depths of 6.5 and 6.6 m below grade, without encountering bedrock.

Results of laboratory testing obtained on this stratum are be summarized in Table 4-3.

Table 4-3
Results of Laboratory Testing
Dense to Very Dense Gravely Sand with Some Silt

Tests	Number of tests	Unit	Results		
			Minimum	Maximum	Average
Grain size analyses (gravel, 5 to 80 mm)	17	%	5.0	57.1	23.1
Grain size analyses (sand, 0.08 to 5 mm)	17	%	27.9	65.5	43.8
Grain size analyses (fines, < 0.080 mm)	17	%	8.1	66.2	33.0
Grain size analyses (clay size, < 0.002 mm)	6	%	3.1	26.4	11.5
Moisture content	17	%	7.8	15.8	10.3

4.3 Bedrock

Bedrock was encountered in all boreholes (BH and W Series) carried out at the site, except in Boreholes BH-401-05, BH-504-05 and BH-506-05. In Borehole BH-401-05, bedrock was not proven, as this last borehole was terminated at a depth of 15.9 m without rock coring and was extended from 15.9 to 20.1 m below existing grade only by cone dynamic penetration tests.

At the site of the West Storage Tank (BH-101-05 to BH-105-05), bedrock was encountered (and proven by core drilling) at depths ranging from 7.7 to 11.1 m below existing ground surface or between (geodetic) elevations 64.6 and 68.1 m. In the vicinity of the West Tank (W-002-04 and W-004-04), bedrock was met at depths of 1.8 and 11.3 m or at elevations 74.6 and 63.9 m respectively.

At the site of the East Storage Tank (BH-106-05 to BH-110-05), bedrock was encountered and proven by core drilling at depths ranging from 5.1 and 7.3 m below existing grade or between elevations 68.8 and 71.3 m. Within a close range of this structure (BH-111A-05, BH-116B-05, BH-117B-05, W-003-04 and W-005-04), bedrock was found at depths ranging from 2.3 to 12.5, or between elevations 63.2 and 75.3 m.

At the site of the proposed LNG Process Area, bedrock was encountered at depths varying from 0.6 to 24.0 m below existing grade at the location of Boreholes BH-501-05 to BH-503-05, and BH-505-05, and Test Pit TP-505-05. Locally at the site of Borehole BH-401-05, bedrock is inferred to be somewhat beyond 20.1 m depth, whereas at the location of Boreholes BH-504-05 and BH-506-05, bedrock is a depths greater than 6.5 and 6.6 m. Within distances of 200 to 300 m from the center of the LNG Process Plant (W-001-04, W-002-04, W-004-04 and W-006-04), bedrock is at depths ranging from 0.2 and 11.3 m below grade.

At the site of the Unloading Lines or at the location of Boreholes BH-301-05 to BH-307-05, and BH-507-05, bedrock was found at depths ranging from 0.6 to 4.6 m below ground surface.

Based on the results of the boreholes, the following general bedrock features were identified at the project site:

- Red and green mudstones or shales;
- Pale to dark grey, greenish grey and black shales or mudstones, with some sandstone, siltstone and calcareous horizons;
- Red mudstones.

The reader should refer to the borehole logs (Appendix I) and also to Section 3.2 of this report for detailed information about the bedrock.

Results of laboratory testing obtained on rock cores are be summarized in Table 4-4. The reader is reminded that the test results included in Table 4-4 are inferred to belong to relatively good quality bedrock since, as stated in Section 2.6.2, compressive strength determinations could not be achieved on very poor to fair quality rock. The compressive strength of the rock varied from 4 to 100 MPa, with average values ranging from 23 to 44 Mpa.

Table 4-4
Results of Laboratory Testing Bedrock

Tests	Number of tests	Unit	Results		
			Minimum	Maximum	Average
Uniaxial unconfined compressive strength on rock cores (not complying to ASTM D 2938) / Boreholes BH-102-05 to BH-110-05 Series and W-002-04 to W-008 Series	24	MPa	3	99	23
Uniaxial unconfined compressive strength on rock cores (complying to ASTM D 2938) / Boreholes BH-505-05 and BH-507-05	6	MPa	12	100	44
Unit weight of rock / Boreholes W-002-04 to W-008-04 Series	12	kN/m^3	26.0	27.2	26.5

Based on the RQD values, the bedrock condition may be summarized as follows within the project site:

- West Storage Tank (Boreholes BH-101-05 to BH-105-05): The bedrock varies from very poor to poor quality down to a depth range of 23 to 38 m below existing grade, and generally becomes good to excellent thereafter.

- East Storage Tank (Boreholes BH-106-05 to BH-110-05, and BH-111A-05): The bedrock varies from very poor to poor quality down to a depth range of 22 to 24 m below grade, and generally becomes good or excellent thereafter.

- LNG Process Plant (Boreholes BH-501-05 to BH-503-05, BH-505-05, W-001-04, W-002-04, W-004-04 and W-006-04): The bedrock varies from very poor to fair quality down to a depth range of 1 to 20 m below grade, and generally becomes good to excellent thereafter.

- Unloading Lines (BH-301-05 to BH-307-05, and BH-507-05): In general the bedrock varies from very poor to fair quality at least down to depths ranging from 3.4 to 6.2 m below existing ground surface. Locally in Borehole BH-302-05, good quality rock is encountered down to 7.3 m depth. In borehole BH-507-05 carried out to 41 m below grade close to the site of a proposed Deep Rock Cut to the jetty, fair to good quality rock was encountered, with also intermittently and locally very poor to poor quality rock between depths of 1.3 to 3.4 m, 12.2 to 14.0 m, 20.3 to 24.5 m and 27.7 to 32.1 m.

In addition to the laboratory testing stated in Table 4-4, petrographic examination for pyrite determination was performed on 20 selected rock cores and 1 split spoon sample (i.e. 18 rock cores from boreholes of the BH Series and 2 rock cores and I split-spoon sample from boreholes of the W Series). The results of this testing are included at the end of Appendix II. They are summarized as follows with respect to the main rock facies, and also in reference to the boreholes and sample depths:

- Black shale (Boreholes BH-102-05 at 10.5 m and W-002-04 at 2.3 m), black shale / light grey mudstone (Borehole BH-102-05 at 12.5 m), and black shale / calcareous sandstone (Borehole BH-102-05 at 11.5 m): with SPPI (Swelling Potential Petrographic Index) values in the range of 39 to 56 and equivalent pyrite contents varying from 0.54 % to 2.60 %, the rock is considered as having a high swelling potential due to the presence of pyrite.

- Light grey, grey or red mudstones: with SPPI (Swelling Potential Petrographic Index) values in the range of 12 to 50 and equivalent pyrite contents varying from 0.02 % to 0.11 %, the rock is considered as stable and is not presenting any swelling potential due to the presence of pyrite.

5. GROUNDWATER CONDITIONS AND CHEMICAL ANALYSES

To allow groundwater level observations, perforated plastic standpipes were installed in most the boreholes of the BH and W Series. Boreholes BH-102-05, BH-104-05, BH-106-05, BH-107-05, BH-110-05, BH-111A-05 and W-004-04 were converted into monitoring wells by the intrusion of a 50 mm diameter PVC and base slotted tube. Borehole BH-503-05 is equipped with a 25 mm size piezometer tube. Boreholes BH-101-05, BH-109-05 and BH-501-05 are provided with a capped bottom PVC liners, and thus cannot serve to monitor the groundwater.

Groundwater was measured in the boreholes of the W Series during Fall of 2004 (6 October 2004). The boreholes of the BH-102-05 through BH-401-05 Series were monitored in early Spring of 2005 (15 April 2005), whereas boreholes of the BH-502-05 to BH-507-05 Series were surveyed for groundwater in Fall of 2005 (14 October 2005). Groundwater levels were thus observed from 1 to 59 days after completion of drilling.

Groundwater readings in the boreholes are summarized on Table 5-1. Artesian conditions were observed in three boreholes (BH-102-05, BH-116B-05, and BH-503-05).

It should be noted that the elevation of the groundwater table is usually not stable as it generally fluctuates with the seasons or subsequent to modifications to the environment. Thus, the groundwater may be found at or very close to the ground surface at certain times of the year, namely during the spring thaw or after periods of heavy precipitations.

Representative groundwater samples were subjected to chemical testing to assess their aggressiveness to concrete. The analytical report for the groundwater is included at the end of Appendix II. The results of chemical analyses performed on water samples retrieved from boreholes of the BH Series are summarized in Table 5-2. An hydrological study of the project site was carried out during the period of July through September 2005 by SNC-Lavalin Environment inc.

Table 5-1
Groundwater Levels

Boreholes	Level of Ground Surface	Date of Groundwater Measurement	Depth of groundwater (below ground surface)	Groundwater Level	Note
	m		M	m	
BH-102-05	75.52	15-Apr-05	-0.20	75.72	Well
BH-103-05	75.67	15-Apr-05	0.22	75.45	Standpipe
BH-104-05	75.72	15-Apr-05	0.46	75.26	Well
BH-105-05	75.41	15-Apr-05	0.20	75.21	Standpipe
BH-106-06	76.25	15-Apr-05	0.51	75.74	Well
BH-107-05	75.79	15-Apr-05	0.07	75.72	Well
BH-108-05	76.19	15-Apr-05	0.33	75.86	Standpipe
BH-110-05	76.20	15-Apr-05	0.39	75.81	Well
BH-111A-05	75.74	15-Apr-05	0.17	75.57	Well
BH-116B-05	75.44	15-Apr-05	-0.35	75.79	Standpipe
BH-117B-05	77.38	15-Apr-05	1.50	75.88	Standpipe
BH-301-O5	76.79	15-Apr-05	0.07	76.72	Standpipe
BH-302-05	77.19	15-Apr-05	1.28	75.91	Standpipe
BH-303-05	71.50	15-Apr-05	0.96	70.54	Standpipe
BH-304-05	66.06	15-Apr-05	0.70	65.36	Standpipe
BH-305-05	64.06	15-Apr-05	0.50	63.56	Standpipe
BH-306-05	61.48	15-Apr-05	0.76	60.72	Standpipe
BH-307-05	52.81	15-Apr-05	3.25	49.56	Standpipe
BH-401-05	76.58	n/a	n/a	n/a	Standpipe
BH-502-05	75.75	14-Oct-05	0.75	75.00	Standpipe
BH-503-05	75.33	14-Oct-05	-0.39	75.72	Piezometer
BH-504-05	75.92	14-Oct-05	0.63	75.29	standpipe
BH-505-05	77.44	14-Oct-05	0.50	76.94	standpipe
BH-506-05	76.77	14-Oct-05	0.40	76.37	standpipe
BH-507-05	54.17	14-Oct-05	4.30	49.87	standpipe
W-001-04	78.14	6-Oct-04	2.50	75.64	standpipe
W-002-04	76.40	6-Oct-04	1.00	75.40	standpipe
W-003-04	77.53	6-Oct-04	2.20	75.33	standpipe
W-004-04	75.15	6-Oct-04	0.64	74.51	well
W-005-04	77.55	6-Oct-04	1.30	76.25	standpipe
W-006-04	79.84	6-Oct-04	0.85	78.99	standpipe
W-008-04	78.60	6-Oct-04	2.74	75.86	standpipe

Table 5-2
Results from Chemical Testing on Groundwater Samples

Parameters	Units	Test Results			
		BH-102-05	**BH-104-05**	**BH-106-05**	**BH-110-05**
pH	pH	8.6	8.8	8.8	11 (doubtful)
Sulfur anion (S=)	mg/L	ND	ND	ND	ND
Alkalinity (Total as $CaCO_3$) pH 4.5	mg/L	320	260	350	200
Bicarbonates (HCO_3 as $CaCO_3$)	mg/L	300	230	320	ND
Carbonate (CO_3 as $CaCO_3$)	mg/L	16	29	29	130
Chloride (Cl)	mg/L	50	180	5.5	5.6
Sulfates (SO_4)	mg/L	11	22	34	36

Notes:

ND = Not detected
N/A = Not applicable
DL = Detection Limit

Based solely on the above results of chemical analyses, i.e. the pH values and sulfates (SO_4), the groundwater is inferred to constitute a low level of aggressiveness to concrete.

However, in recent years it has become a general and common practice to recommend the use of sulphate resistant Type 50 cement, or cement with silica fume, for concrete foundations installed in the St. Lawrence River Low Lands. This recommendation should be implemented on the Rabaska Project, in order to avoid long term sulfatation of the concrete foundations and floor-slab-on-grades, as the process is initiated by two sources: the low sulfate content in the groundwater, and the sulfate generation from the oxidation of the pyrite enclosed in the dark shale beddings within the mudstone and shale bedrock.

6. CONCLUSIONS AND RECOMMENDATIONS

6.1 General

Rabaska Limited Partnership is contemplating the construction of a LNG Receiving Terminal, in Levis, Quebec. The project is located close to the eastern limit of the Municipality of Levis, between the south shore of the St. Lawrence River and a main project area situated some 1.4 km to the southeast or between Hydro-Quebec power lines and Highway 20 (see Drawing T-1050-B-0000-4GDD-0001 in Appendix III).

Based on available information, the project comprises the construction of the following industrial components and facilities:

- ***LNG Storage Tanks;***
- ***LNG Process Area;***
- ***Unloading Lines (Deep Rock Cut);***
- ***Access Road and Paved Areas.***

Comments and preliminary recommendations, from a geotechnical point of view, are included in the following sections of this report, to assist in the early design of earthworks and foundations, slab-on-grades, roads and paved zones, and lay-down area, within the proposed facilities. Geotechnical recommendations given herein will likely need to be adjusted and reviewed in the future, as conceptual details and project features and layout get more advanced. Dynamic response of foundations will also be addressed in the future within the detailed engineering phase.

6.2 LNG Storage Tanks

6.2.1 Structural Features and Subsurface Conditions

As indicated on Drawing T-1050-B-0000-4GDD-0001 (Appendix III), the construction of two LNG Storage Tanks are contemplated at the site of Boreholes BH-101-05 to BH-110-05. The tanks will be 90 m in diameter and some 46 m in height (with dome). Each tank will be installed within a 150 m x 150 m permanent depressed enclosure with its base at a depth of approximately 10 m below the existing ground surface, or at about elevation 65 m. Center and edge static bearing pressures induced by the storage tanks are expected respectively not exceed 170 and 280 kPa during operation, or 250 and 320 kPa during initial water testing.

The storage tanks will be seated on circular mat type foundations. The mat foundations will be provided with thermal insulation and underlying heating cables, to avoid the formation of permafrost due to the extreme low temperature of the LNG containment.

Subsurface investigations recently carried out at the sites of the LNG Storage Tanks indicated the following soil and bedrock conditions:

- ***West Storage Tank:*** Under about 0.2 m of topsoil, compact sand and gravel was encountered down to about 1.4 to 2.3 m below grade, locally followed by firm to very stiff clayey soils extending to 2.7 or 4.2 m depth. These are underlain, at depths ranging from 1.7 to 4.1, by dense to very dense gravely silt and sand that extends to 7.7 or 11.1 m below existing ground surface, where bedrock was encountered. Bedrock, found at about elevations 64.6 to 68.1 m, generally consisted (Boreholes BH-101-05 to BH-104-05) of very poor to poor quality light grey mudstone or (locally) sandstone, with about 5 to 10 % layers of greenish grey and dark grey shale. Locally (BH-105-05), very poor to fair quality red mudstone was encountered.

- ***East Storage Tank:*** Under about 0.5 to 0.9 m of topsoil or 0.6 m of peat, compact (and locally dense) sand and gravel or sand and silt with some or trace of gravel was encountered down to about 1.5 to 3.3 m below grade, locally followed by stiff to very stiff clayey soils extending to 3.7 or 4.9 m depth. These are underlain, at depths ranging from 2.7 to 4.9 m by dense to very dense gravely sand with some silt and occasional cobbles and boulders, or silty and gravely sand that extends to 5.1 or 7.3 m below existing ground surface, where bedrock is met. Bedrock, found at about elevations 68.8 to 71.4 m, generally consisted (BH-106-05 to BH-108-05) of very poor to fair quality dark grey shale, grey mudstone with 10-15 % of black shale layers, or greenish grey shale or mudstone with black shale layers. Locally (BH-109-05 and BH-110-05), very poor to good quality red mudstone, with dark shale layers was identified.

Groundwater was observed (15 April 2005) between 0 and 0.5 m depth. Groundwater levels are subject to seasonal fluctuations and modifications to the environment.

6.2.2 Excavation

The proposed Storage Tanks will be installed within two depressed enclosures, extending to a depth of about 10 m below the existing ground surface.

Based on the known subsurface conditions, and in view of the general design schemes and layouts already contemplated at the project site, the following comments and preliminary recommendations are given concerning the excavation of the depressed enclosures of the LNG Storage Tanks:

(1) In view of the soil stratigraphy prevailing at the site, and the high groundwater conditions observed namely at thaw, permanent side slopes of 2.5 (H) : 1 (V) within the generally well graded and dense to very dense soils are considered to be at risk of experimenting seasonal or intermittent slope erosion and sand/silt sloughing problems. In this perspective, the surface and the toe of the slope within the overburden should be covered and protected with an inverted granular filter pad. This granular slope protection and pad will consist of a geotextile fabric covered by coarse clean crushed stone, eventually combined with inclined perforated drains located a short distance from the exposed face of the slopes. This feature should be implemented with a drainage system installed at the crown of slope to intercept runoff. Furthermore, a drainage trench should be provided a short distance "upstream" of the toe of the slope, as to allow systematic draw-down of the water table, and thus prevent toe erosion. At the toe of the overburden slope, a minimum 1.5 m wide horizontal berm should be provided at the bedrock surface.

(2) Although the bedrock encountered at the site within the proposed excavation depths was generally found to vary from very poor to fair quality, further to generally being thin bedded, it is believed that excavation thereof will require hydraulic breaking process and blasting locally. Ripping is not considered practical, except maybe locally in the very poor to poor rock surface (often most severely fractured and weathered). The reader is reminded that bedrock at the site of Trial Excavation TE-B-05 (Section 3.2.2.3) could be easily excavated at

least down to about 2 m below the bedrock surface by means of an excavator equipped with a 1.7 m^3 size bucket.

(3) Permanent pumping will be required in the excavations due to the high groundwater conditions prevailing at the project site. This situation was observed in October 2005 in Trial Excavation TE-B-05 (Section 3.2.2.3) where pumping was also needed to draw-down the water table at the bottom of the excavation. Recommended drainage features at the bottom of the depressed enclosures consist of a layer of clear and uniform crushed stone entrapped between unwoven geotextile fabrics and traversed by perforated pipes. This system should be connected to sumps.

(4) In view of the inclination of the bedrock beddings and the bedrock layered structure, the permanent excavated rock surface should generally be provided with a side slope of 1 (H) : 1 (V). Exposed rock surfaces will undergo alteration and weathering due to the exposition to air, and frost action. To limit the consequences of long term rock weathering and alteration and retain future rock scaling, the excavated and exposed rock surfaces should be covered with a steel wire mesh connected to closely spaced anchors. As the bedrock is likely dipping in one direction, i.e. towards the southeast, rock bolting may also be locally contemplated. Where dipping would permit it, steeper rock slopes close to 2.5 (V) : 1 (H) and generally no higher than 6 m may be contemplated if duly protected from weathering and alteration by shotcrete covered wire mesh adequately retained by closely spaced rock anchors, and provided with underdrainage.

(5) Exposed bedrock, at or below the foundation level, should be protected against alteration and weathering due to oxidation and/or frost action. In this perspective it should be sealed without delay by means of a thin layer (100 mm) of concrete. The concrete sealing should be placed immediately after excavation and after approval by a qualified technician, of the exposed rock surfaces. Also rock surfaces (under foundations) should be protected against frost action, if the excavations are carried out during winter.

(6) In addition to the above recommendations, and to prevent the rock swelling process that could be initiated by the oxidation of the dark mudstone and/or

shale beddings of the bedrock, the exposed bedrock bearing surface under foundation should be permanently maintained at least 0.5 m below groundwater. This could therefore require deeper excavation within the bedrock, with respect to what was needed to comply to recommendations stated in (5). In this perspective, drainage within the depressed enclosures of Storage Tanks should be achieved accordingly.

(7) Because of inferred and locally proven swelling properties and weathering characteristics, the excavated rock shall not be reused as fill material under structures or floor-slabs-on-grade.

6.2.3 Foundations

The LNG Storage Tanks may be founded on circular mat type foundations, in compliance to the following recommendations:

(1) An allowable net bearing pressure of 500 kPa is recommended for large size mat foundations seated on the poor quality bedrock, or on a layer of concrete in direct contact with the bedrock. This allowable bearing pressure was determined essentially from judgment based on the bedrock descriptions performed at the foundation levels on the rock cores and also at shallow depth in the trial excavations. As the frequently encountered very poor to poor quality bedrock is thinly bedded and shows closely spaced discontinuities, it may be considered in many instances to be as good or better than a thoroughly compacted and well interlocked granular fill. The above recommended allowable bearing pressure is consistent with suggested lower bearing values stated in typical building codes for comparative poor quality rock (schist, slate and shale). As the center and edge static bearing pressures induced by the storage tanks are expected respectively not exceed 170 and 280 kPa during operation, or 250 and 320 kPa during initial water testing, the aforementioned allowable bearing pressure is deemed sufficient.

(2) For transient loadings due to wind or seismic events, the above net allowable bearing pressure may be increased by 30 percent.

(3) As the dark grey and black mudstone / shale will likely show swelling properties in addition to be prone to undergo quick alteration and weathering process when exposed to air, it is recommended in zones where this type of rock is encountered, that bedrock be excavated to comply to recommendations stated in 6.2.2. (Items 5 and 6). This is aimed at permanently maintaining the exposed rock (under the tank) below groundwater to avoid oxidation that could generate swelling and weathering. All excavated and exposed rock surfaces should then be immediately covered with fill materials, or with concrete to limit alteration and weathering, as outlined in Section 6.2.2. (Item 6).

(4) Based on a freezing index of about 1250°C. days typical for the Quebec City / Levis area, the mat foundation of the tanks should normally be provided with a soil cover of at least 2.0 m at its periphery. It is inferred that frost action may take place at the outset of the said mat foundation during winter, even in consideration of the partially heated environment provided (by heating cable) below the tank bottom. If required, a portion of the soil cover may be replaced by a suitable thickness of thermal insulation adequately and horizontally placed at shallow depth at the periphery of the mat foundation.

(5) Due to the very low temperature of the LNG containment within the Storage Tanks, thermal insulation and heating cables are essential to prevent the formation of permafrost within the bearing bedrock.

6.3 <u>LNG Process Area</u>

6.3.1 Structural Features and Subsurface Conditions

The proposed LNG Process Area will comprise equipment and compressor foundations, footings to support lightly loaded steel columns, and small buildings. These will likely be situated within a distance of 500 m southwest of the West Storage Tank.

The area was investigated by means of Boreholes BH-401-5 and BH-501-05 to BH-506-05 and Test Pits TP-503-05 to TP-505-05. Compact sand with gravel and silt was encountered generally down to depths of about 0.8 to 6.1 m, followed by dense to very dense sand with gravel and occasional cobbles and boulders. Bedrock was found at depths ranging from 5.2 to 13.2 m (BH-501-05 and BH-502-05), or beyond 6.5 or

6.6 m (BH-504-05 and BH-506-05) or at depth greater than 20.1 m (BH-401-05 and BH-503-05), whereas it was also locally identified at shallow depths i.e. between 0.6 or 0.8 m (TP-505-05 and BH-505-05). Bedrock varies from very poor to poor quality and consists of light grey or siltstone or mudstone. Soil and bedrock data and properties are summarized in Table 6-1, together with recommended design parameters.

Table 6-1
Soil and Bedrock properties and parameters (LNG Process Area)

Typical Soil or Bedrock Strata			**Properties**			**Recommended Parameters**						
Soil / Bedrock Type	**Depth Range**	**Stratum Thickness**	**Unit Weight γ**	**N_{SPT} Values**	**q_u (6)**	**c'**	**ϕ'**	**E**	**Poisson Ratio ν**	**Ko**	**Ka**	**Kp**
	m	**m**	**KN/m^3**	**Blows / 0.3 m**	**MPa**	**kPa**	**(°)**	**MPa**	**-**	**-**	**-**	**-**
Topsoil (2)	0 to 0.15 or 0.30	0.2 to 0.3	18 to 20	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a
Compact sand with silt and gravel (2)	0.2 to 0.8 or 6.1	0.6 to 6.0	20 to 21	13 to 30 (1)	n/a	0	30	10 to 28	0.35	0.5	0.30	3.3
Dense to very dense sand with gravel and occasional cobbles (3)	1.2 or 4.6 to 6.1+	1.5 to 4.9+	21.5 to 22.3	34 - 60+ refusal	n/a	0	34	95 to 200	0.30	0.5	0.28	3.6
Very severely and fractured bedrock (4)	0.8 to 1.5	0.7	26.0	refusal	n/a	0	36	150 to 300	0.30	n/a	0.26	3.8
Poor to good quality bedrock (4)	1.5 to 4.7+	3.2+	26.5	n/a	12 to 40	n/a	42	5000 to 20000	0.30	n/a	n/a	n/a
Fair to good quality bedrock (5)	5.2+	0.9+	26.5	n/a	20 to 50	n/a	42	10000 to 25000	0.30	n/a	n/a	n/a

Notes :
1 - Excluding the first 0.5 m depth
2 - General soil feature (BH-401-05, and BH-501-05 to BH-506-05)
3 - Frequent soil feature (BH-401-05, BH-502-05 to BH-504-05, and BH-506-05)
4 - Local bedrock feature (BH-505-05)
5 - Local bedrock feature (BH-502-05)
6 - Rock unconfined compressive strength

A temporary Lay-down Area is contemplated in the vicinity of Boreholes BH-503-05 to BH-505-05 and Test Pits TP-503-05 to TP-505-05. Subsurface investigation works performed in this area, indicated the presence of 0.1 to 0.3 m of topsoil followed by silty sand with some gravel in a compact state of relative density at depths smaller than 1.6 to 2.0 m (BH/TP-503-05 and BH/TP-504-05) and locally (BH/TP-504-05) comprising a thin clayey silt layer. Locally at the site of BH/TP-505-05 rock is encountered at 0.6 or 0.8 m below existing grade.

Groundwater was observed (14 October 2005) between 0 and 0.5 m depth. Groundwater levels are subject to seasonal fluctuations and modifications to the environment.

6.3.2 Foundations

Isolated or strip shallow footings may be contemplated for structures and equipments within the proposed LNG Process Area. They should be designed and constructed to comply to the following recommendations:

(1) Based on the borehole results and the soil N_{SPT} index values, the net allowable bearing pressures applicable to shallow footings are given in Table 6-2. These may be considered for footings seated on intact natural soils, in view of limiting total settlements to less than 20 mm, and differential settlements to less than 15 mm between adjacent columns typically spaced at 5 m or more (to limit angular distortion to 1:330). Static spring constant for foundation are given in Table 6-3.

(2) Footings seated on (poor quality) bedrock may be designed for an allowable net bearing pressure of 500 kPa. The bedrock surface should be cleared of severely fragmented rock, prior to footing installation.

(3) For transient loadings due to wind or seismic events, the above allowable bearing pressures may be increased by 30 percent.

(4) All topsoil, peat and organic soils, as well as remoulded, soft or frozen soils and uncontrolled fill should be excavated prior to the installation of a footing on intact natural soils.

(5) Protection and sealing of exposed rock should be carried out as outlined in Section 6.2.2. and in Section 6.2.3. Item (1).

(6) To limit differential movements for cases where some of the footings belonging to a same structure would be partly seated on rock, and on fill or soil, a well compacted and non swelling crushed stone cushion, at least 300 mm in thickness, should be provided under any footing that otherwise would have been on rock.

(7) The minimum width of footings (on soil or on rock) should be 1 m. Settlements are expected to be less than 3 to 5 mm for shallow foundations seated on bedrock, whereas settlements of the order of 15 to 20 mm may occur with footings installed on compacted granular fill (seated on rock), or on compact to dense natural soils.

(8) Footings (on soil or rock) exposed to freezing conditions must be protected against frost action. Based on a freezing index of about 1250 C°· days typical for the Quebec City / Levis area, exterior foundations of heated structures should be provided with a soil cover of at least 2.0 m. For unheated structures, the soil cover should be increased to 2.4 m. Thermal insulation may be contemplated to reduce frost penetration during winter.

Table 6-2
Allowable Bearing Pressure for Footings seated on Natural Soil or on Bedrock

<table>
<tr><th colspan="2">Depth (m)</th><th colspan="2">Approximate elevation (m)</th><th colspan="7">Net allowable bearing pressure (kPa)</th></tr>
<tr><th>from</th><th>to</th><th>From</th><th>to</th><th>Borehole BH-401-05</th><th>Borehole BH-501-05</th><th>Borehole BH-502-05</th><th>Borehole BH-503-05</th><th>Borehole BH-504-05</th><th>Borehole BH-505-05</th><th>Borehole BH-506-05</th></tr>
<tr><td>0</td><td>1.0</td><td>76.3</td><td>75.3</td><td>30</td><td>60</td><td>70</td><td>100</td><td>50</td><td>200</td><td>60</td></tr>
<tr><td>1.0</td><td>2.0</td><td>75.3</td><td>74.3</td><td rowspan="4">300</td><td>170</td><td rowspan="3">270</td><td rowspan="2">200</td><td>200</td><td rowspan="4">500 (on bedrock)</td><td rowspan="4">250</td></tr>
<tr><td>2.0</td><td>3.0</td><td>74.3</td><td>73.3</td><td rowspan="3">200</td><td rowspan="3">300</td></tr>
<tr><td>3.0</td><td>5.2</td><td>73.3</td><td>71.1</td><td rowspan="2">300</td></tr>
<tr><td>5.2</td><td>6.1</td><td>71.1</td><td>70.2</td><td>500 (on bedrock)</td></tr>
</table>

Table 6-3
Proposed Static Spring Values for Footings seated on Natural Soil or on Bedrock

<table>
<tr><th colspan="2">Depth (m)</th><th colspan="2">Approximate elevation (m)</th><th colspan="7">Approximate range of modulus of subgrade reaction (MN/m^3)</th></tr>
<tr><th>from</th><th>to</th><th>From</th><th>to</th><th>Borehole BH-401-05</th><th>Borehole BH-501-05</th><th>Borehole BH-502-05</th><th>Borehole BH-503-05</th><th>Borehole BH-504-05</th><th>Borehole BH-505-05</th><th>Borehole BH-506-05</th></tr>
<tr><td>0</td><td>1.0</td><td>76.3</td><td>75.3</td><td>2 – 3</td><td>5 – 8</td><td>6– 9</td><td>6 - 14</td><td>3 - 7</td><td>13 - 20</td><td>4 - 9</td></tr>
<tr><td>1.0</td><td>2.0</td><td>75.3</td><td>74.3</td><td rowspan="4">27 – 42</td><td>11 – 21</td><td rowspan="3">20 - 40</td><td rowspan="2">13 - 25</td><td>13 - 20</td><td rowspan="4">50 – 120 (on bedrock)</td><td rowspan="4">25 - 40</td></tr>
<tr><td>2.0</td><td>3.0</td><td>74.3</td><td>73.3</td><td rowspan="3">13 – 20</td><td rowspan="3">20 - 38</td></tr>
<tr><td>3.0</td><td>5.2</td><td>73.3</td><td>71.1</td><td rowspan="2">25 - 50</td></tr>
<tr><td>5.2</td><td>6.1</td><td>71.1</td><td>70.2</td><td>50 - 120 (on bedrock)</td></tr>
</table>

6.3.3 Floor Slabs-on-grade

The following recommendations are applicable to floor slabs-on-grade, within heated structures:

(1) All organic, remoulded, soft or frozen soils, and uncontrolled fill materials should be excavated prior to the installation of a floor slab-on-grade.

(2) The floor slabs-on-grade should be underlain by at least 300 mm of crushed stone of size 20-0 mm compacted to at least 95% of the Modified Proctor maximum dry density. The crushed stone must be tested prior to placement (in accordance with the SPPI procedure - Standard NQ 2560-500) to ensure that it is non-swelling. This crushed stone pad combined with the underlaying soil is believed to yield a subgrade modulus ranging from 25 to 40 MN/m^3.

(3) Construction joints should be provided in the floor slabs-on-grade especially at the face of foundation walls and columns, to allow small differential settlements to occur without damage.

(4) Constant vibratory loadings applied to floor slabs warrant detailed dynamic analysis, which are considered outside the scope of this mandate.

6.3.4 Temporary Lay-down Area

The proposed temporary Lay-down Area will be provided with a granular fill cover to enable the free traffic of heavy delivery vehicles, further to allow the storage, handling and assembling of structural components, specialized industrial equipment, and temporary facilities needed during the construction and commissioning of the LNG Receiving Terminal.

In view of the existing subsurface and groundwater conditions, the following recommendations are provided:

(1) All topsoil, peat, large roots and stumps, loose or soft soil, or soil remolded by the construction activities should be removed prior to the placement of the granular fill cover within the contemplated lay-down area. At this stage of the site preparation, peripheral drainage ditches should be provided at the outset of the lay-down areas and the exposed bottom of the stripped area should, whenever possible, be profiled towards the ditches.

(2) The granular fill cover within the contemplated lay-down area should consist of the following sub-base and base materials:

- Sub-base (crushed or granular material, fragmented rock): 500 mm
- Base course (crushed stone): 400 mm

The sub-base material should essentially be well draining and consist of well graded crushed or granular material (typically complying to MG 112 of Ministère des Transports du Québec) with less than 10 % of fines particles smaller than 0.080 mm. Excavated rock fragmented to less than 50 - 200 size particles could also be used as sub-base material if adequately rolled with a dozer.

The base course materials should comply to MG 20 gradation (Ministère des Transports du Québec) or be composed of MG 56 materials topped with MG 20, with less than 8 % of fines particles smaller than 0.080 mm.

(3) Periodic profiling and maintenance of the finished base course surface is essential during the service life of the temporary lay-down area, to maintain traficability and surface runoff evacuation towards the ditches.

6.4 Unloading Lines

6.4.1 Structural Features and Subsurface Conditions

Unloading conduits are contemplated between the south shore of the St. Lawrence River and the LNG Storage and Process Facilities, at the site of Borehole BH-507-05, and Boreholes BH-307-05 to BH-301-05.

Sand in loose to compact state of compactness and with silt was generally found between ground surface and a depth of approximately 0.6 to 1.5 m in most of the above boreholes, whereas locally (BH-302-05) loose sand extends to 3.1 below grade. Also locally (BH-301-05) a stiff to very stiff clay stratum was encountered down to 0.9 m below grade. In Boreholes BH-302-05 to BH-305-05, the loose soils are underlain by compact to very dense sand extending to 2.4 or 4.6 m below ground surface. Bedrock was proven at shallow depths of 0.6 to 0.9 m (BH-301-05, BH-307-05 and BH-507-05) and at depths ranging from 1.3 to 4.6 m below grade (BH-302-05 to BH-306-05).

Groundwater was observed (15 April 2005) in Boreholes BH-301-05 to 306-05 between 0 and 1.0 m depth, whereas in Boreholes BH-307-05 and BH-507.05, groundwater was found (April and October 2005) at depths ranging from 3.3 to 4.3 m. Groundwater levels are subject to seasonal fluctuations and modifications to the environment.

6.4.2 Foundations

The following allowable net bearing pressures may be considered for footings seated at depths of about 2.0 to 2.4 m below grade:

- Borehole BH-301-05: 250 kPa on very severely fractured bedrock
- Borehole BH-302-05: 30 kPa on soil (or 200 kPa on soil at 3.1 m)
- Borehole BH-303-05: 100 kPa on soil (or 500 kPa on rock at 2.4 m)
- Borehole BH-304-05: 100 kPa on soil (or 500 kPa on rock at 2.7 m)

- Borehole BH-305-05: 500 kPa on bedrock
- Borehole BH-307-05: 500 kPa on bedrock
- Borehole BH-507-05: 500 kPa on bedrock

Settlements are expected to be less than 3 to 5 mm for shallow foundations seated on bedrock, whereas settlements of the order of 10 to 20 mm may occur for footings installed on intact natural soils. Differential settlements of adjacent footings seated on soils may range from 8 to 15 mm thus limiting angular distortion to 1:350 between columns typically spaced at 3 to 5 m or more.

Recommendations, already provided in Sections 6.2.2, 6.2.3. and 6.3.2 concerning the minimum width of footings, and the protection against rock alteration to air and swelling process, and frost protection are applicable to the installation of the Unloading Lines, if they are routed or founded at shallow depth in the overburden or within the upper layer of bedrock.

6.4.3 Deep Rock Cut to the Jetty

Between the south shore of the St. Lawrence River and a point located at least some 100 m SE of the locus of Boreholes BH-307-05 and BH-507-05 and Trial Excavation TE-A-05, a 300 m long open Rock Cut with a maximum depth of 22 m is presently contemplated for (i) LNG unloading lines originating from the ship docking facilities, and (ii) an access road. The Rock Cut will be oriented in a NW-SE direction.

Based on the local geology and in reference to the results of the subsurface investigations, that are valid only near the southeastern part of the Rock Cut, the bedrock consists of a succession of relatively thin (10 to 300 mm thick) mudstone and siltstone beds with shale interbeds (10 mm thick). In general the rock beddings are dipping from 58 to 62°. However, from local direct visual observations performed at shallow depth in Trail Excavation TE-A-05, the orientations of the beddings were found to be at 30°. Although the bedding orientations observed in TE-A-05 are considered favorable with respect to the lateral rock stability of the proposed deep linear rock cut aligned in a NW to SE direction, the local geology also provides indications that unfavorable bedding orientations would also exist.

In view of the above, the following general comments are provided:

(1) According to the usual practice and the standards of Ministère des Transports du Québec, which are normally applied to road construction, rock cuts to a maximum depth of 6 m into sedimentary rocks would normally be provided with 1.0 (H) to 2.5 (V) rock slopes. Deeper rock cuts, may warrant special geological studies. As a general rule, pending more detailed studies, the rock cuts should be provided with 6 m wide benches at vertical intervals of no more than 12 m. A rock catch ditch is also recommended for falling rocks alongside the roadway.

(2) In view of the above standards, several benches will be required with the contemplated deep and steep-sloping Rock Cut. Furthermore, with due consideration the poor quality of the rock, which is thinly bedded and could locally also be dipping unfavorably, the extensive weathering and scaling process which will likely be worsen by frost action and seasonal groundwater seepage remains a major concern, with permanently exposed rock slopes, even if they are no steeper than 1.0 (H) to 2.5 (V).

(3) In this perspective and as the project gets more advanced towards developing open Rock Cut schemes, slope protection by means of wire mesh, rock bolting, rock mass drainage, etc,. will need to be addressed and optimized from a rock mechanic point of view.

(4) The bedrock encountered at the site (Borehole BH-507-05) within the proposed excavation depth of the Deep Rock Cut was found to vary from poor to excellent quality, further to generally being thin bedded. It is believed that excavation thereof will require hydraulic breaking process or blasting. Ripping is not considered practical, except maybe locally in the very poor to poor rock encountered at shallow depths.

6.4.4 Rock Cut on the cliff near the Jetty

The steep rocky cliff along the south shore of the St. Lawrence River near the proposed Jetty appears to be the site of only local and limited surface rock scaling. Seismic events would not aggravate the situation.

It is presently envisaged to route the LNG unloading lines originating from the ship docking facilities on a sustaining vertical steel frame structure. This structure, which will be founded on a granular fill placed at the toe of the cliff, shall reach at least the top of the 21 m high steepest section (2.3V:1.0H) of the cliff. In this perspective, rock excavation in the immediate vicinity of the sustaining structure should comply to the general recommendations already outlined in Section 6.4.3.

6.5 Access Roads and Paved Areas

The following recommendations apply to the design and construction of access roads and paved areas:

(1) All topsoil, peat, uncontrolled fill, loose or soft soil, or soil remolded by the construction activities should be removed prior to the construction of access roads and paved areas.

(2) For local traffic of heavy and light vehicles, the following sub-base, base and pavement layers may be considered:

- Heavy vehicles:
  - Sub-base (sand or crushed stone) : 300 mm
  - Base course (crushed stone) : 450 mm
  - Wearing course (asphalt) : 55 mm (EB-14) + 40 mm (EB-10S)
- Light vehicles:
  - Sub-base (sand or crushed stone) : 300 mm
  - Base course (crushed stone) : 300 mm
  - Wearing course (asphalt) : 60 mm (EB-10S)

(3) All materials should comply with the Ministère des Transports du Québec (MTQ) standards, in terms of gradation, soundness and compaction. The above EB-14 and EB-10S pavement denominations are as per MTQ Standards.

(4) To ensure proper performance of the access roads and paved areas, all sub-base and base course materials should be implemented at the sub-grade.

(5) Long term performance and good behavior of access roads paved areas rely on good drainage. Therefore, adequate drainage ditches should be provided to maintain groundwater below the base course and sub-base layers.

6.6 Soil and Bedrock Dynamic Properties

The dynamic parameters of the overburden soil and of the bedrock were determined on-site by means of down-hole seismicity surveys recently carried out by Geophysics GPR International Inc. in Boreholes BH-101-05, BH-109-05, and BH 501-05. The detailed results of the testing are gathered and discussed in Appendix VIII of this report, and are summarized hereafter in Table 6-4.

Table 6-4
Dynamic Parameters of Soils and Bedrock

Borehole	Depth	Description	Assumed Mass Density	Poisson Ratio		Gs		Es	
				min.	max.	min.	max.	min.	max.
	m		kg/m^3	-		GPa		GPa	
BH-101-05	2.2 - 9.7	Dense to very dense sand with some gravel and silt	1 900	0.36	0.46	0.46	0.87	1.30	2.41
	9.7 - 23.4	Poor to good quality mudstone	2 600	0.33	0.48	0.91	3.93	2.71	11.31
BH-109-05	2.3 - 5.1	Compact to dense sand and silt	1 900	n/a	n/a	0.14	0.18	n/a	n/a
	5.1 - 7.3	Very severely fractured and weathered red mudstone	1 900	n/a	n/a	0.39	0.79	n/a	n/a
	7.3 - 20.8	Fair to good quality red and greenish grey mudstone	2 600	0.40	0.48	0.87	1.90	2.53	5.52
	20.8 - 23.2	Fair to good quality grey shale	2 600	0.43	0.45	2.45	3.80	7.14	10.85
BH-501-05	0.0 - 2.1	Loose to compact silty sand, some gravel	1 900	n/a	n/a	n/a	n/a	n/a	n/a
	2.1 - 6.1	Compact silt and sand, trace of gravel	1 900	0.37	0.46	0.14	0.63	0.39	1.81
	6.1 - 13.2	Dense to very dense silt and sand, trace of gravel and clay	1 900	0.40	0.47	0.44	0.73	1.25	2.11
	13.2 - 19.8	Very poor to fair quality limestone	2 600	0.42	0.45	1.51	2.46	4.36	7.05

6.7 Ground Apparent Electrical Resistivity

Vertical electrical soundings were carried out in October 2005 by Geophysics GPR International Inc. to determine the ground apparent electrical resistivity for grounding purposes. The electrical soundings were performed at two locations (RT-1-05 and RT-2-05) using the Wenner four electrode array. The procedure and the detailed results of the in-situ testing are provided in Appendix VIII.

6.8 General Conditions and Limitations

The use of this report is subjected to the following General Conditions and Limitations, Sections A through F, applicable to geotechnical report:

A. USE OF THE REPORT

A.1 The factual data, interpretations and recommendations contained in this report pertain to a specific project as described in the report and are not applicable to any other project or site location. If the project is modified in concept, location or elevation or if the project is not initiated within eighteen months of the date of the report TERRATECH should be given an opportunity to confirm that the recommendations are still valid.

A.2 The recommendations given in this report are intended only for the guidance of the design engineer. The number of test holes to determine all the relevant underground conditions which may affect construction costs, techniques and equipment choice, scheduling and sequence of operations would normally be greater than has been carried out for design purposes. Contractors bidding on, or undertaking the work, should rely on their own investigations, as well as their own interpretations of the factual test hole data, as to how subsurface conditions may affect their work.

B. FOLLOW-UP

B.1 All details of the design and proposed construction may not be known at the time of submission of TERRATECH's report. It is recommended that TERRATECH be retained during the final design stage to review the design drawings and specifications related to foundations, earthworks, retaining systems and drainage, to determine that they are consistent with the intent of TERRATECH's report.

B.2 Retention of TERRATECH during construction is recommended to confirm and document that the subsurface conditions throughout the site do not materially differ from those given in TERRATECH's report and to confirm and document that construction activities did not adversely affect the design of TERRATECH's recommendations.

C. SOIL AND ROCK CONDITIONS

C.1 Soil and rock descriptions in this report are based on commonly accepted methods of classification employed in professional geotechnical practice. Classification and identification of soil and rock involves judgement and TERRATECH does not guarantee descriptions as exact, but infers accuracy only to the extent that is common in current geotechnical practice.

C.2 The soils and rock conditions described in this report are those observed at the time of the study. Unless otherwise noted, those conditions form the basis of the recommendations in the report. The condition of the soil and rock may be significantly altered by construction activities (traffic, excavation, pile driving, blasting, etc.) on the site or on adjacent sites. Excavation may expose the soils to changes due to wetting, drying or frost. Unless otherwise indicated the soil and rock must be protected from these changes or disturbances during construction.

D. LOGS OF TEST HOLES AND SUBSURFACE INTERPRETATIONS

D.1 Soil and rock formations are variable to a greater or lesser extent. The test hole logs indicate the approximate subsurface conditions only at the location of the test holes. Boundaries between zones on the logs are often not distinct, but rather are transitional and have been interpreted. The precision with which subsurface conditions are indicated depends on the method of boring, the frequency of sampling, the method of sampling and the uniformity of subsurface conditions. The spacing of test holes, frequency of sampling and type of boring also reflect budget and schedule considerations.

D.2 Subsurface conditions between test holes are inferred and may vary significantly from conditions encountered at the test holes.

D.3 Groundwater conditions described in this report refer only to those observed at the place and time of observation noted in the report. These conditions may vary seasonally or as a consequence of construction activities on the site or adjacent sites.

E. CHANGED CONDITIONS

Where conditions encountered at the site differ significantly from those anticipated in this report, either due to natural variability of subsurface conditions or construction activities, it is a condition of the use or reliance by the client of this report that TERRATECH is notified of the changes and provided with an opportunity to review the recommendations of this report. Recognition of changed soil and rock conditions requires experience and it is recommended that an experienced geotechnical engineer be employed to visit the site with sufficient frequency to detect if conditions have changed significantly.

F. DRAINAGE

Drainage of subsurface water is commonly required either for temporary or permanent installations for the project. Improper design or construction of drainage can have serious consequences. TERRATECH can take no responsibility for the effects of drainage unless TERRATECH is specifically involved in the detailed design and follow-up site services during construction of the system.

7. REFERENCES

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8. PERSONNEL

The subsurface investigations (boreholes and test pits) were carried out under the close supervision of Mr. Hugues Chouinard, Mr. Alain Périard, and Mr. Denis Désaulniers, Senior Technicians of Terratech. The trial excavations into bedrock were performed in the presence of Mr. Jean-Jacques Hébert, Geologist, and Mr. Yves Boulianne, Eng.

The detailed description of the recovered rock cores was performed by Mr. Christian Boucher, Geologist, Mrs. Isabelle Robillard, Geologist, and Mr. Martin Labelle, Geologist in training, and by Mr. Alain Blanchette, Geologist, M.A.Sc.

Sections 1 to 3.1, and 4 to 8 of this report were prepared by Mr. Raymond Bousquet, Eng., M.A.Sc. Section 3.2 of the report was written by Mr. Jean-Jacques Hébert, Geologist, and Mr. Yves Boulianne, Eng. The document was reviewed (ISO Conformity) by Mr. Henri Madjar, Eng., M.A.Sc.

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