Qualitas

SNC-LAVALIN

GEOTECHNICAL ENGINEERING REPORT

Québec City Armoury
Québec (Québec)

Project #: 09-010-010

November 27, 2014

Qualitas

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Geotechnical Engineering Report

Québec City Armoury
Québec (Québec)

Project #: 09-010-010

Distribution: Mr. Stéphane Digonnet (1 copie, 1 PDF)
Project Manager

GROUPE QUALITAS INC. November 27, 2014

2014-11-28

Ronald Blackburn, Eng.
Director- National Capital
OIQ member #: 41430

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1.0 INTRODUCTION

Groupe Qualitas Inc. (hereafter referred to as Qualitas) was mandated to carry out a site investigation by Mr. Stéphane Digonnet, project manager for SNC-Lavalin, at the Québec Armoury, in Québec City.

This report presents our geotechnical study for the design of appended buildings. The construction details being unknown at the moment of writing, the level of study and recommendations suits the current design stage. The study objectives are to provide general recommendations, to identify and review key geotechnical issues, and to provide a basis to develop construction cost estimates.

The report encloses in order of appearance, site and project description, investigation methodology, ground conditions, geotechnical recommendations and bearing capacities.

This geotechnical report was written for SNC-Lavalin in order to achieve the previously described objectives. We ask for Qualitas to be informed of any modification to the project in order to revalidate the recommendations stated in this report.

2.0 GEOTECHNICAL INVESTIGATION SUMMARY

2.1 GEOTECHNICAL FIELD WORK

Geotechnical investigation works were carried out on September 30th and October 1st, 2009. The scope of work included 9 boreholes. The boreholes were completed using a trailer-mounted Mobil-Drill B-53 rig. The field work was conducted under the supervision of a geotechnical field inspector from Qualitas.

Boreholes F-09-01 to F-09-09 were advanced using continuous-flight augers. In each borehole, remolded samples were recovered using a standardized split-spoon sampler (outside diameter of 51 mm and length of 610 mm). Standard Penetration Tests (SPT) were performed simultaneously in compliance with the ASTM D1586 Standard, recording blow counts for each 150 mm increment and calculating the N value from the mid-300 mm over a 610 mm total penetration. Remolded samples were collected all the way down to the bedrock.

Rock core samples were recovered using NQ-caliber diamond core barrels. A general description of the rock was noted by the technician/geologist and *Recuperation* and *Rock Quality Designation* (RQD) were measured for each sample.

Table 1 shows the depth reached in every boreholes as well as the elevation of bedrock.

Table 1
Reached Depth in Boreholes at Site

Borehole	Surface elevation(m)	Depth of borehole / bottom elevation (m)	Depth/elevation (m)	
			Disintegrated rock	Bedrock
B-09-03	98.81	4.27 / 94.54	0.60/98.21	1.35/97.46
B-09-04	96.15	5.18 / 90.97	1.35/94.80	1.80/94.35
B-09-05	97.01	5.84 / 91.17	3.00/94.01	3.15/93.86
B-09-06	95.84	6.05 / 89.79	--	3.00/92.84
B-09-07	98.24	6.10 / 92.14	1.20/97.04	3.00/95.24
B-09-08	98.81	4.47 / 94.34	1.30/97.51	1.60/97.21

The borehole logs are provided in Appendix 2, at the end of this report.

2.2 INSTRUMENTATION

A standpipe piezometer was installed in every borehole to measure the groundwater level in the overburden. These piezometers consisted of a 19 mm diameter flexible “carlon” pipe installed at the bottom of the borehole.

2.3 SURVEYING

The borehole locations were designated on the field by Qualitas in accordance with the demands of SNC-Lavalin.

The survey was done by the client (SNC-Lavalin). Elevations are arbitrary.

The locations of every borehole are shown on the overall site plan (drawing # 09-010-010-10) in Appendix 1.

3.0 LABORATORY TESTING

All soil samples and rock core samples were brought back to Qualitas laboratory in Québec City, to be described and classified. Laboratory tests were performed on selected samples. The complete laboratory testing program is presented at Table 2.

Table 2
Laboratory Tests

Sample	Depth (m)	Test
B-09-06, SS-2	0.60 to 1.20	Grain Size Analysis
B-09-04, RC-5	2.60 to 2.80	Uniaxial Compressive Strength
B-09-08, RC-8	3.80 to 4.00	Uniaxial Compressive Strength

The results are presented in Appendix 3.

Two samples were submitted to an accredited lab in order to analyze the swelling potential of bedrock (known as the Swelling Potential Petrographic Index – SPPI). For both samples, the index is equal to 50, which is greater than 10. In compliance with BNQ 2560-500/2003 Standard, chemical analyses were performed to confirm the swelling potential of the rock. Table 3 shows the Swelling Potential Petrographic Index. The detailed results are as well presented in Appendix 3.

Table 3
Petrographic Index Swelling Potential

Sample	Depth (m)	SPPI
B-09-06, RC-6	3.40	50
B-09-03, RC-4	1.75	50

The unused samples will be stored for a period of six months after the publication date of this report (2009-10-21). Unless directed otherwise by SNC-Lavalin, the samples will be destroyed once this time period has elapsed.

4.0 GROUND CONDITIONS

4.1 GENERAL

Subsurface conditions encountered at specific locations are shown on the borehole logs enclosed in Appendix 2 and are discussed below. The general soil profile observed in boreholes is reasonably similar, with expected variations in soil composition and units thickness.

In the following sections, the soil description has been interpreted and simplified to major strata for the purpose of geotechnical analysis. The soil profile is presented in descending order.

Table 4 shows the overall stratigraphy observed at the site.

Table 4
Stratigraphy

Borehole #	B-09-03	B-09-04	B-09-05	B-09-06	B-09-07	B-09-08
Description	**Depth Limits/Thickness (m)**					
Asphalt concrete	0.00-0.10 0.10	0.00-0.10 0.10	0.00-0.10 0.10	0.00-0.10 0.10	0.00-0.02 0.02	0.00-0.10 0.10
Concrete	---	---	---	---	0.02-0.30 0.28	---
Crushed gravel (20-0)	0.10-0.20 0.10	0.10-0.50 0.40	0.10-0.20 0.10	0.10-0.50 0.40	---	0.10-0.25 0.15
Fill: Sand, some silt, traces of gravel.	0.20-0.60 0.40	0.50-0.90 0.40	---	0.50-3.00 2.50 (1)	---	0.25-0.45 0.20
Fill: Silty sand with variable proportion of gravel. Presence of scraps : brick, wood, mortar, etc.	---	0.90-1.35 0.45	0.20-3.00 2.80	---	0.30-1.20 0.90 (2)	0.45-1.30 0.85
Disintegrated argillaceous limestone	0.60-1.35 0.75	1.35-1.80 0.45	3.00-3.15 0.15 (3)	---	1.20-3.00 1.80	1.30-1.60 0.30
Bedrock : argillaceous limestone	1.35-4.27 2.92	1.80-5.18 3.38	3.15-5.84 2.69	3.00-6.05 3.05	3.00-6.10 3.10	1.60-4.47 2.87

Note: (1) Presence of blackish sand, cobbles and asphalt debris
(2) Presence of rock fragments under the concrete slab.

4.2 ASPHALT CONCRETE AND CRUSHED GRAVEL (20-0)

In most boreholes, a 100 mm thick surficial layer of asphalt concrete is observed. At borehole B-09-07, a thinner 25 mm-thick layer of asphalt is present. In this borehole, a 280 mm thick concrete slab underlies the asphalt layer.

In boreholes B-09-03 to B-09-06 and in borehole B-09-08, we observed a 100 to 400 mm thick layer of crushed gravel (20-0 mm) under the asphalt concrete.

4.3 FILL

In boreholes B-09-03, B-09-04, B-09-06 and B-09-08, a fill layer composed of sand with some silt and traces of gravel underlies the crushed gravel layer. The fill is respectively 0.40, 0.40, 2.50 and 0.20 m thick. At borehole B-09-06, a blackish layer of sand containing cobbles and asphalt debris is observed within the fill.

In boreholes B-09-04, B-09-05, B-09-07 and B-09-08 at depths of 0.90, 0.20, 0.30 and 0.45 m, a fill composed of silty sand with various gravel content is observed. This fill was encountered up to 1.35, 3.00, 1.20 and 1.30 m of depth. Various debris were also noted in the fill such as brick, wood chips and mortar. In borehole B-09-07, fragments of calcareous rock were encountered under the concrete slab.

The compactness of the fill layer varies from loose to compact.

4.4 BEDROCK

Table 1 and 4 show depths and corresponding elevations at which the bedrock was encountered in each borehole. In every borehole, except borehole B-09-06, the upper part of the bedrock is disintegrated and likened to a silty soil (or, in the case of borehole B-09-05, a sand and gravel). The thickness of the disintegrated rock layer varies from 0.15 to 1.80 m. Better quality rock is encountered at depths between 1.35 to 3.15 m. The recovered rock is described as an argillaceous limestone. The bedding angle of the rock mass is 30° to the core axis. Calcite veinlets are also observed throughout. Some calcite-filled fractures were noted. Open fractures with a sandy/silty or chlorite infilling were also observed. In boreholes B-09-03, B-09-04, B-09-06 and B-09-07, the limestone is very fractured at shallow depth, and becomes less fractured with increasing depth.

Results of compression strength test on rock core samples vary from 29.0 to 69.7 MPa. According to the *Classification or Rock with Respect to Strength* (after Marinos and Hoek, 2001) proposed by the Canadian Foundation Engineering Manual, these UCS values correspond to grade R3 to R4 rock (medium strong to strong). The compressive test data sheets are presented in Appendix 3.

4.5 GROUND WATER

Groundwater levels were measured in the piezometer installed in every borehole. The depth and elevation of the groundwater table at different test locations are shown in Table 5.

Table 5
Ground Water Level

Test Location	Depth (m)	Elevation (m)
B-09-03	2.10	96.71
B-09-04	1.93	94.22
B-09-05	3.79	93.22
B-09-06	2.65	93.19
B-09-07	0.74	97.50
B-09-08	2.15	96.66

It should be noted that groundwater levels can change according to climatic conditions and that they are subjected to seasonal variations or local disturbances (nearby trench, pumping, well, etc.)

5.0 GEOTECHNICAL RECOMMENDATIONS

5.1 FOREWORDS

Conclusion and geotechnical recommendations of this investigation are based on the necessary assumption that the data recovered in the boreholes are representative of the overall soil and rock conditions prevailing at the site.

5.2 BEARING CAPACITY

Based on the current geotechnical investigation, shallow foundations appear well suited to the site conditions. Since bedrock was encountered at shallow depth and based on the rock cored samples, it is likely that the foundations will be supported by unweathered rock underlying the disintegrated rock layer. Depending on the embedment depth of the footings, these will be constructed either on highly fractured rock or good quality, much less fractured rock. Two options, presented below, are therefore available for consideration by the designer:

5.2.1 Shallow foundation on highly fractured rock

The net allowable bearing capacity for foundations on highly fractured rock is 500 kPa.

5.2.2 Shallow foundation on good quality rock

Considering instead that the highly fractured rock layer is excavated and that the footings are placed on moderately fractured to good quality rock, the net allowable bearing capacity will be 3000 kPa.

5.2.3 Settlements

Settlements will remain below the usually admitted values of 25 mm (total) and 19 mm (differential) for applied loads generating stresses inferior to the bearing capacities of sections 5.2.1 and 5.2.2.1

5.3 FOUNDATION LAYOUT

In any case, rock foundations should be constructed on a clean, flat surface. Weathered rock and loose fragments should be excavated under the supervision of skilled technician. A lean concrete mix or crushed gravel pad compacted at 98% of the Modified Proctor can be used to create a uniform surface.

5.4 SLAB-ON-GRADE

The following recommendations are common practice for the design of a slab-on-grade structurally independent from the building.

- Remove the topsoil or any existing fill to expose the highly fractured rock and/or intact rock (the weathered, disintegrated soil-like rock layer must also be excavated for it has a high swelling potential).

- Replace excavated soils to base level with a granular fill constructed with MG 112 sand. Lifts shall not be thicker than 300 mm and compacted to 95% of the material maximum dry density, as defined by the Modified Proctor method of compaction (hereafter referred to as MPMDD);
- The base course thickness will be designed to meet the required bearing capacity for the slab. The base will be constructed with MG 20 crushed gravel (as described in NQ 2560-114), in lifts less than 300 mm thick, compacted to 98% MPMDD. Crushed gravel shall be certified according to BNQ-2560-500 and BNQ-2560-510 Standards.

5.5 FOUNDATION PROTECTION INCLUDING SLAB-ON-GRADE AGAINST ROCK SWELLING

The Swelling Potential Petrographic Index was determined on two rock core samples (step 1 of BNQ-2560-500 Standard), and were classified with a medium to high SPPI. In step 2, chemical analyses (total sulfur and SO_4) were performed on samples to determine pyrite content. Table 6 shows test results as well as the calculated pyrite content. Based on the latter, the rock mass swelling potential is extremely high.

Table 6
Chemical Analysis (Rock Swelling Potential)

Sample	Depth (m)	SPPI	Total Sulfur (%)	SO_4 (mg/kg)	Pyrite (%)
B-09-06 RC-6 SG-12981	3.40	50	0.83	127	1.54
B-09-03 RC-4 SG-12982	1.75	50	1.43	179	2.67

Considering the extremely high swelling potential of the bedrock, one shall protect the rock as soon as it is exposed (a delay not exceeding 12 hours is recommended) with a lean concrete layer or a bituminous membrane of appropriate thickness. The recommended construction sequence during rock excavation is to excavate in the morning and to apply protection in the afternoon.

5.6 ROCK ANCHORS

Where rock anchors are required, we recommend installing injection rock anchors. Rock anchors have the following modes of failure:

- failure of the steel tendon or top anchorage;
- failure of the grout/tendon bond;
- failure of the rock/grout bond;
- failure within the rock mass.

The Qualitas anchor design procedure is provided in Appendix 4. The design parameters are provided in Table 7.

<table>
<tr><th colspan="4">Table 7
Rock Anchors Design Parameters</th></tr>
<tr><th rowspan="2">Failure mode</th><th colspan="3">Parameters</th></tr>
<tr><th>Symbol</th><th>Description</th><th>Design Value</th></tr>
<tr><td>Failure of the steel tendon/top anchorage</td><td>σ_y</td><td>Tensile strength of anchorage</td><td>See manufacturer specs</td></tr>
<tr><td>Tendon/grout failure</td><td>f'_c</td><td>Uniaxial compressive strength of grout</td><td>As specified (usually 30 MPa at 28 days)</td></tr>
<tr><td rowspan="4">Rock/grout failure</td><td>f'_c</td><td>Uniaxial compressive strength of grout</td><td>As specified (usually 30 MPa at 28 days)</td></tr>
<tr><td>q_u</td><td>Uniaxial compressive strength of rock</td><td>30 MPa</td></tr>
<tr><td rowspan="2">L_{s2}</td><td>Minimum embedment length</td><td>3 m in the rock mass</td></tr>
<tr><td>Recommended embedment vs RQD</td><td>80 X borehole diameter</td></tr>
<tr><td rowspan="2">Rock mass failure</td><td>β</td><td>Failure cone apex half angle</td><td>40^o</td></tr>
<tr><td>γ</td><td>Unit weight of rock</td><td>26 kN/m^3</td></tr>
</table>

For each site, a minimum of two test anchors should be proof-loaded to 1.33 times the working (service) load. The test procedure is described in CFEM (2006), section 26.12.4.8.

5.7 STABILITY OF EXCAVATIONS AND DEWATERING

During construction, the walls of trenches in the overburden or the rock should be sloped in accordance with the *Code de sécurité pour les travaux de construction* of the Commission sur la Santé et la Sécurité au Travail (CSST). At any time, the contractor will be responsible for the stability of the excavations. However, the following guidelines can be used by the designer to estimate excavation volume and cost.

With adequate groundwater control, excavations above water table should have walls sloped at 1.0H:1.0V or flatter. Below water level, excavations should be sloped at 2.0H: 1.0V. In the unweathered rock, excavations should be sloped at 1.0H :7.0V. Wall stability in the rock being dependent on the joints orientation, the inclination of the walls should be reduced if unfavorable joint sets are encountered. Scaling of rock faces should also be done from the top down before any worker is allowed at the bottom. Wall stability should be assessed by a trained geologist.

The excavation walls should be inspected regularly for any signs of instability.

Excavated materials and heavy vehicles should not be allowed at distance less than one time the excavation depth from the edge of the excavation.

Excavation of fills and disintegrated rock should be made using a standard hydraulic excavator. However, excavation of better quality rock may be difficult with this equipment. When encountered, a high performance excavator, jackhammer or blasting technique could be necessary for advancing to the required depth.

Excavations should not, in any case, cause damage to other existing structures, buried pipes, etc. If required, retaining systems should be used.

Surface runoffs should be controlled by sloping the ground away from the excavation, constructing dikes around the top of the trench or with permanent or temporary collection ditches. When excavation under the groundwater table is necessary, the contractor will have to provide an adequate dewatering system. Collected water should be disposed of in accordance with local requirements.

5.8 PROTECTION AGAINST FROST HEAVE

In order to provide sufficient frost protection, all exterior footings (and interior footings in unheated buildings) should be embedded 1.80 m deep.

Alternatively, rigid board insulation could be used to protect the foundations (STYROFOAM High Load or equivalent). The thermal insulation should be designed in accordance with the Canadian Foundation Engineering Manual (CFEM 2006), Article 13.5.2 or by the manufacturer's specifications.

5.9 BACKFILLING OF FOUNDATIONS

The exterior of the foundation walls should be backfilled with a free draining, non-frost susceptible granular fill meeting the specifications of a MG 112 sand, as described in BNQ Standard NQ 2560-114. The backfill material shall be placed in lifts not exceeding 300 mm and compacted at 90% of the MPMDD.

To prevent structural damages to the foundation wall, backfilling should be done simultaneously on both sides of the wall. The backfill level difference between the inside and the outside should not exceed 600 mm.

The previous comment being a general recommendation, backfill material should be chosen and placed according to projected nearby utilities (i.e. parking, sidewalk, access path, etc.).

5.10 PERMANENT DRAINAGE

A perforated drain recovered with a geotextile and surrounded by clean gravel should be installed at the foundation level.

The top layer should be constructed with a less permeable material and sloped away from the foundations, to minimize runoff infiltration along the wall.

Rainwater coming from the gutter shall be redirect far from the foundation wall.

Based on the slab-on-grade final design and ground water conditions, sealing of the foundation may have to be considered by the designer.

5.11 REUSE OF THE EXCAVATED MATERIALS

Crushed gravel (20-0) and sand fill containing some silt and traces of gravel may be reused as class B material according to the *Cahier des charges et devis généraux (*CCDG 2009).

Silty sand fill material will not be reusable as class B due to the presence of multiple debris.

Excavated rock material will not be reusable based on their high swelling potential (see section 5.5).

In any case, excavated materials such as fill and soils will only be reusable for exterior planning where installations like roads, parking, buried pipes, sidewalk, etc. are not planned.

6.0 QUALITY CONTROL

Quality control should be done in order to assess the following:

- soil conditions;
- trench bottom for the construction of the foundation;
- drainage;
- backfilling material and compaction;
- concrete mix, etc.

7.0 SCOPE AND LIMITATIONS OF THE REPORT

This geotechnical report was written for SNC-Lavalin and the project engineering consultants in order to achieve the previously described objectives. We ask for Qualitas to be informed of any modification to the project in order to revalidate the recommendations stated in this report.

APPENDIX 1

DRAWING

Projet:	
QUEBEC CITY ARMOURY	
Titre: BOREHOLES LOCATION	Client: SNC-LAVALIN

LÉGENDE

Forages

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N/DOSSIER :	09-010-010
DATE :	2014-11-24
N/DESSIN :	09-010-010-01
	1/1

APPENDIX 2

GEOTECHNICAL LOGS

Qualitas

BOREHOLE LOG

CLIENT : SNC-Lavalin
PROJECT : Geotechnical Investigation
LOCATION : Québec City Armoury, Québec
FILE : 09-010-010

BOREHOLE : B-09-03

DATE : 2009-09-30
COORDINATES :
E : **N :**
AZIMUT : ° **PLUNGE :** °

DEPTH (m)	ELEVATION (m) Arbitrary	DESCRIPTION	WATER LEVEL 2009-10-09	SAMPLES: TYPE AND NUMBER	SAMPLES: CONDITION	SAMPLES: RECOVERY (%)	N or RQD (%)	WATER CONTENT AND ATTERBERG LIMITS (%)	OTHER TESTS	S_u (kPa) / S_{us} (kPa) / S_r (kPa) / S_{rs} (kPa) / N_{dc} (blows/300 mm)
	98.81									
0.10	98.71	Asphalt concrete.								
0.20	98.61	Crushed gravel (20-0).								
		Fill: Brown sand, some silt and trace gravel. Dense.		CF-1		50	34			
0.60	98.21	Bedrock: Disintegrated argillaceous limestone. Likened a silty soil. Very dense.		CF-2		68			N: >50	
				CF-3		100			N: >50	
1.35	97.46	Argillaceous limestone. Fractured up to 1.75 m deep. Bedding angle at 30° from the core axis. Open fractures filled with calcite, chlorite and/or sandy and silty soils.	2.10 m	CR-4		100	40		Swelling potential (SPPI)	
				CR-5		100	85			
				CR-6		100	75			
				CR-7		100	95			
4.27	94.54	**End of borehole at 4.27 m depth.**								

REMARKS :

DRILLING METHOD :

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BOREHOLE LOG

CLIENT : SNC-Lavalin
PROJECT : Geotechnical Investigation
LOCATION : Québec City Armoury, Québec
FILE : 09-010-010

BOREHOLE : B-09-04
DATE : 2009-10-01
COORDINATES :
E : **N :**
AZIMUT : ° **PLUNGE :** °

DEPTH (m)	ELEVATION (m) Arbitrary	DESCRIPTION	WATER LEVEL 2009-10-01	TYPE AND NUMBER	CONDITION	RECOVERY (%)	N or RQD (%)	WATER CONTENT AND ATTERBERG LIMITS (%) W_P W W_L (20 40 60 80)	OTHER TESTS	▲ S_u (kPa) ▽ S_{us} (kPa) ★ S_r (kPa) S_{rs} (kPa) ○ N_{dc} (blows/300 mm) (20 40 60 80)
	96.15									
0.10	96.05	Asphalt concrete.								
		Crushed gravel (20-0).		CF-1		55			N: >50	
0.50	95.65	Fill: Brown sand, some silt and traces of gravel.								
0.90	95.25	Heterogeneous fill: Brown silty sand, some gravel. Presence of debris: brick, mortar. Compact.		CF-2		62	25			
1.35	94.80	Bedrock: Disintegrated argillaceous limestone. Likened a silty soil. Very dense.	1.93 m	CF-3		87	5			
1.80	94.35	Argillaceous limestone. Highly fractured up to 2.55 m deep. Bedding angle at 30° from the core axis. Open fractures filled with calcite, chlorite and/or sandy and silty soils.		CR-4		100	45			
				CR-5		100	85		f'=69.7 MPa	
				CR-6		100	80			
5.18	90.97	**End of borehole at 5.18 m depth.**								

REMARKS :

DRILLING METHOD :

Qualitas

BOREHOLE LOG

CLIENT : SNC-Lavalin
PROJECT : Geotechnical Investigation
LOCATION : Québec City Armoury, Québec
FILE : 09-010-010

BOREHOLE : B-09-05

DATE : 2009-09-30
COORDINATES :
E : N :
AZIMUT : ° PLUNGE : °

DEPTH (m)	ELEVATION (m) Arbitrary	DESCRIPTION	WATER LEVEL 2009-10-09	SAMPLES: TYPE AND NUMBER	CONDITION	RECOVERY (%)	N or RQD (%)
	97.01						
0.10	96.91	Asphalt concrete.					
0.20	96.81	Crushed gravel (20-0).					
		Heterogeneous fill: Brown silty sand, traces to some gravel. Presence of debris: brick, mortar, etc. Loose to compact.		CF-1		61	17
				CF-2		42	11
				CF-3		0	6
				CF-4		17	13
				CF-5		92	5
3.00	94.01	Bedrock: Disintegrated argillaceous limestone. likened a silty soil.		CR-6		90	
3.15	93.86	Argillaceous limestone. Highly fractured up to 4.40 and 4.80 m deep. Bedding angle at 30° from the core axis. Open fractures filled with calcite, chlorite and/or sandy and silty soils.	3.79 m	CR-7		100	75
				CR-8		100	80
5.84	91.17	**End of borehole at 5.84 m depth.**					

IN SITU AND LABORATORY TESTS

WATER CONTENT AND ATTERBERG LIMITS (%)	OTHER TESTS	▲ S_u (kPa) ▽ S_{us} (kPa) ★ S_r (kPa) S_{rs} (kPa) ○ N_{dc} (blows/300 mm)
W_P — W — W_L; 20 40 60 80		20 40 60 80

REMARKS :

DRILLING METHOD :

Qualitas

BOREHOLE LOG

CLIENT : SNC-Lavalin
PROJECT : Geotechnical Investigation
LOCATION : Québec City Armoury, Québec
FILE : 09-010-010

BOREHOLE : B-09-06

DATE : 2009-09-30
COORDINATES :
E : **N :**
AZIMUT : ° **PLUNGE :** °

DEPTH (m)	ELEVATION (m) Arbitrary	DESCRIPTION	WATER LEVEL 2009-10-09	SAMPLES: TYPE AND NUMBER	SAMPLES: CONDITION	SAMPLES: RECOVERY (%)	N or RQD (%)	WATER CONTENT AND ATTERBERG LIMITS (%) (W_P, W, W_L) 20 40 60 80	OTHER TESTS	▲ S_u (kPa) ▽ S_{us} (kPa) ★ S_r (kPa) S_{rs} (kPa) ○ N_{dc} (blows/300 mm) 20 40 60 80
	95.84									
0.10	95.74	Asphalt concrete.								
		Crushed gravel (20-0).		CF-1		66	46			
0.50	95.34	Heterogeneous fill: Brown sand with some silt, traces of gravel. Blackish sand locally. Presence of cobbles, asphalt and debris. Compact to dense.		CF-2		71	92		G	
				CF-3		85	40			
				CF-4		54	20			
			2.65 m	CF-5		83	56		refus	
3.00	92.84	Bedrock: Argillaceous limestone. Highly fractured up to 3.65 m deep. Bedding angle at 30° from the core axis. Open fractures filled with calcite, chlorite and/or sandy and silty soils.		CR-6		100	0		Total sulfur SPPI	
				CR-7		100	55			
				CR-8		100	80			
				CR-9		100	70			
6.05	89.79	**End of borehole at 6.05 m depth.**								

REMARKS :

DRILLING METHOD :

BOREHOLE LOG

CLIENT : SNC-Lavalin
PROJECT : Geotechnical Investigation
LOCATION : Québec City Armoury, Québec
FILE : 09-010-010

BOREHOLE : B-09-07

DATE : 2009-10-01
COORDINATES :
E : **N :**
AZIMUT : ° **PLUNGE :** °

DEPTH (m)	ELEVATION (m) Arbitrary	DESCRIPTION	WATER LEVEL 2014-10-09	TYPE AND NUMBER	CONDITION	RECOVERY (%)	N or RQD (%)	WATER CONTENT AND ATTERBERG LIMITS (%) W_P W W_L (20 40 60 80)	OTHER TESTS	S_u (kPa), S_{us} (kPa), S_r (kPa), S_{rs} (kPa), N_{dc} (blows/300 mm) (20 40 60 80)
	98.24	Asphalt concrete.								
0.30	97.94	Concrete slab.								
		Heterogeneous fill: Brown silty sand with some gravel. Rock fragments under the concrete slab). Presence of concrete debris. Loose.	0.74 m	MA-1						
				CF-2		28	9			
1.20	97.04	Bedrock: Disintegrated argillaceous limestone. Likened a silty soil. Very dense to dense.		CF-3		21	23			
				CF-4		71	66			
				CF-5		50			N: >50	
3.00	95.24	Argillaceous limestone. Highly fractured up to 3.70 m deep. Bedding angle at 30° from the core axis. Open fractures filled with calcite, chlorite and/or sandy and silty soils.		CR-6		100	0			
				CR-7		100	70			
				CR-8		100	85			
6.10	92.14	**End of borehole at 6.10 m depth.**								

REMARKS :

DRILLING METHOD :

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Qualitas

BOREHOLE LOG

CLIENT : SNC-Lavalin
PROJECT : Geotechnical Investigation
LOCATION : Québec City Armoury, Québec
FILE : 09-010-010

BOREHOLE : B-09-08

DATE : 2009-09-30
COORDINATES :
E : **N :**
AZIMUT : ° **PLUNGE :** °

DEPTH (m)	ELEVATION (m) Arbitrary	DESCRIPTION	WATER LEVEL 2009-10-09	SAMPLES: TYPE AND NUMBER	SAMPLES: CONDITION	SAMPLES: RECOVERY (%)	IN SITU AND LABORATORY TESTS: N or RQD (%)	WATER CONTENT AND ATTERBERG LIMITS (%) W_P W W_L (20 40 60 80)	OTHER TESTS	▲ S_u (kPa) ▽ S_{us} (kPa) ★ S_r (kPa) S_{rs} (kPa) ○ N_{dc} (blows/300 mm) (20 40 60 80)
	98.81									
0.10	98.71	Asphalt concrete.								
0.25	98.56	Crushed gravel (20-0).								
		Fill: Brownish sand with some silt, traces of gravel up to 0.45 m deep. Blackish silty and gravelly sand. Presence of cobbles, woods chips and mortar debris. Dense.		CF-1		28	28			
				CF-2		78	68			
1.30	97.51	Bedrock: Disintegrated argillaceous limestone. Likened a silty soil. Dense.		CF-3		80				
				CF-4		0			N: >50	
1.60	97.21	Argillaceous limestone. Highly fractured up to 2.25 and 2.55 m deep. Bedding angle at 30° from the core axis. Open fractures filled with calcite, chlorite and/or sandy and silty soils.	2.15 m	CR-5		100	55			
				CR-6		100	0			
				CR-7		100	50			
				CR-8		100	60		f'=29.0 MPa	
4.47	94.34	**End of borehole at 4.47 m depth.**								

REMARKS :

DRILLING METHOD :

APPENDIX 3

LABORATORY TEST RESULTS

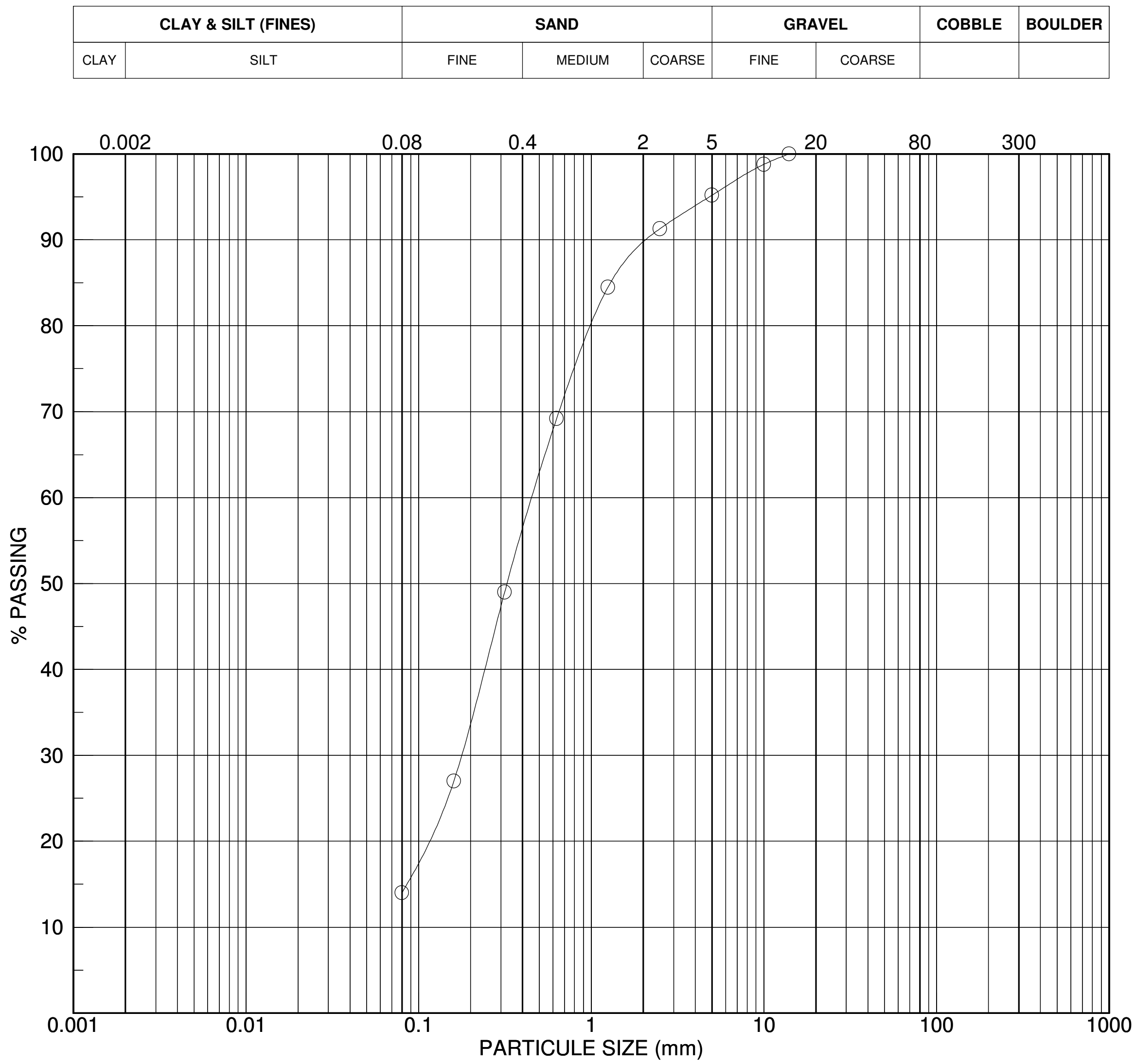
Section 1: Grain Size Distribution Curve
Section 2: Uniaxial Compressive Strength
Section 3: Swelling Potential of Rock
  i) Geologic Report
  ii) Chemical Analyses

SECTION 1

GRAIN SIZE DISTRIBUTION CURVE

PARTICULE SIZE DISTRIBUTION

PROJECT : Geotechnical Investigation, Québec City Armoury
LOCATION : Québec (Québec)
FILE : 09-010-010



	Boring	Éch.	Depth (m)	Description	Gravel (%)	Sand (%)	Clay & Silt (%)
○	F09-06	CF-2	0.60 - 1.20	Sand, some silt, traces of gravel	5	81	14.0

SECTION 2

UNIAXIAL COMPRESSIVE STRENGTH

Qualitas

6155, rue des Tournelles
Québec (Québec) G2J 1P7
Tél : (418) 626-5211
Fax : (418) 626-9312

BÉTON DE CIMENT (CAROTTE)
ESSAIS DE RÉSISTANCE À LA COMPRESSION - CSA A23.2-14C

Soumis à : M. Stéphane Digonnet , dir. de projets
SNC-LAVALIN INC.
455, rue René-Lévesque Ouest
Monréal, Québec, H2Z 1Z3

Dossier N° : 09-010-010
Date : 2009-10-15

Entrepreneur : -
Projet : Manège Militaire
Localisation : Québec (Québec)

Fournisseur de béton : -
Partie bétonnée : Forage : carotte de roc
Localisation du prélèvement : F 09-04 : 2,60 à 2,80 mètres
Date du prélèvement : 2009-09-30
Formule de mélange : -

SPÉCIFICATIONS DU BÉTON

Résistance (MPa) : -
Type(s) de ciment : -
Granulat max. (mm) : -
Affaissement (mm) : -
Air entraîné (%) : -
Temp. du béton (°C) : -
Adjuvant(s) : -

CARACTÉRISTIQUES DU BÉTON LIVRÉ

N° de billet : - **N° du camion** : -
Volume dans camion (m³) : -
Heure de chargement : - **de prélèvement** : 12:00
Temp. (°C) abri : - **extérieur** : -
Affaissement (mm) : - **reprise** : -
Affaissement après l'ajout d'un superplastifiant (mm) : -
Air entraîné (%) : - **reprise** : -
Temp. béton (°C) : - **reprise** : -
Masse volumique (kg/m³) : -
Qté d'eau ajoutée (l) : -
Ajout d'adjuvant(s) : - **qté** : -

Technicien : Client

Type de moules : Carotte Mesurées indiv.
Temp. 1er 24 h (°C) : - min. - max.
Entreposage : -
Transporteur : -

	Date	Heure
Départ du chantier :	-	-
Arrivée au laboratoire :	2009-10-08	12:00

Échantillonné selon la norme CSA A23.2

RÉSISTANCE À LA COMPRESSION

Éprouvette numéro	Diamètre moyen (mm)	Hauteur (mm)	H / D	Date de l'essai	Âge (heures ou jours)	Facteur de correction	Résistance à la compression (MPa)
09-B-10722	47,2	94,3	2,00	2009-10-15	15 jours	1,00	**69,7**

REMARQUES : L'âge de la carotte correspond à la date du forage

Chargé de projet : François Blanchet
François Blanchet,ingénieur junior

17:26

Qualitas

6155, rue des Tournelles
Québec (Québec) G2J 1P7
Tél : (418) 626-5211
Fax : (418) 626-9312

BÉTON DE CIMENT (CAROTTE)
ESSAIS DE RÉSISTANCE À LA COMPRESSION - CSA A23.2-14C

Soumis à : M. Stéphane Digonnet , dir. de projets
SNC-LAVALIN INC.
455, rue René-Lévesque Ouest
Monréal, Québec, H2Z 1Z3

Dossier N° : 09-010-010
Date : 2009-10-15

Entrepreneur : -
Projet : Manège Militaire
Localisation : Québec (Québec)

Fournisseur de béton : -
Partie bétonnée : Forage : carotte de roc
Localisation du prélèvement : F 09-08 : 3,80 à 4,00 mètres
Date du prélèvement : 2009-09-30
Formule de mélange : -

SPÉCIFICATIONS DU BÉTON

Résistance (MPa) : -
Type(s) de ciment : -
Granulat max. (mm) : -
Affaissement (mm) : -
Air entraîné (%) : -
Temp. du béton (°C) : -
Adjuvant(s) : -

CARACTÉRISTIQUES DU BÉTON LIVRÉ

N° de billet : - **N° du camion** : -
Volume dans camion (m³) : -
Heure de chargement : - **de prélèvement** : 12:00
Temp. (°C) abri : - **extérieur** : -
Affaissement (mm) : - **reprise** : -
Affaissement après l'ajout d'un superplastifiant (mm) : -
Air entraîné (%) : - **reprise** : -
Temp. béton (°C) : - **reprise** : -
Masse volumique (kg/m³) : -
Qté d'eau ajoutée (l) : -
Ajout d'adjuvant(s) : - **qté** : -

Technicien : Client

Type de moules : Carotte Mesurées indiv.
Temp. 1er 24 h (°C) : - **min.** - **max.**
Entreposage : -
Transporteur : -

	Date	Heure
Départ du chantier :	-	-
Arrivée au laboratoire :	2009-10-08	12:00

Échantillonné selon la norme CSA A23.2

RÉSISTANCE À LA COMPRESSION

Éprouvette numéro	Diamètre moyen (mm)	Hauteur (mm)	H / D	Date de l'essai	Âge (heures ou jours)	Facteur de correction	Résistance à la compression (MPa)
09-B-10750	47,2	94,4	2,00	2009-10-15	15 jours	1,00	**29,0**

REMARQUES : L'âge de la carotte correspond à la date du forage

Chargé de projet : François Blanchet
François Blanchet,ingénieur junior

SECTION 3

SWELLING POTENTIAL OF ROCK

i) Geologic Report
ii) Chemical Analyses

i) GEOLOGY REPORT

Qualitas

6155, Rue des Tournelles, Québec (Québec) G2J 1P7
Tél. : 418-626-5211 Fax : 418-626-9312 www.lsblic.ca

CALCUL DE L'INDICE PÉTROGRAPHIQUE DU POTENTIEL DE GONFLEMENT (IPPG) BNQ 2560-500

No de l'échantillon : 09-SG-12981
Type de matériau : Roc (carotte)
Provenance : Forage F-09-06, profondeur 3,4 m.
Source première : Forage F-09-06, profondeur 3,4 m.
Usage proposé : Roc au niveau du fond d'excavation
Date de prélèvement : 2009-10-07
Prélevé par : François Blanchet, ing. Jr.
Firme : Groupe Qualitas inc.

Projet : Manège Militaire Québec, Québec
N. / no de dossier : 09-10-010
V. / no de dossier : -
V. / no d'échantillon : -
Rapport soumis à : Digonnet Stéphane, dir. de projet
Firme : SNC-Lavalin
455, rue René-Lévesque Ouest (Monréal)

FACIÈCES PÉTROGRAPHIQUE	Indice IP	FRACTION GRANULOMÉTRIQUE											
		20 mm et plus *			10/14 mm			5/10 mm			2,5/5 mm		
		g (± 0.1)	% (± 1)	IP x % (± 0,1)	g (± 0.1)	% (± 1)	IP x % (± 0,1)	g (± 0.1)	% (± 1)	IP x % (± 0,1)	g (± 0.1)	% (± 1)	IP x % (± 0,1)
Calcaire argileux	0,50	1000	100	50,0									
Total		1000	100										

* *Roc. L'analyse granulométriue n'est pas requise.*

	20 mm et plus	10/14 mm	5/10 mm	2,5/5 mm
A Sommation IP x % pour chaque fraction granulométrique	50,0	0,0	0,0	0,0
B Pourcentage pondéré de la fraction granulométrique retenue sur le tamis	100			
C Résultat de A x B/100 (Ci)	50,0			

IPPG cumulatif global pondéré (Σ Ci) : 50

Intérprétation des indices pétrographiques de potentiel de gonflement (IPPG) :

0 - 10 : NÉGLIGEABLE | 11 - 20 : FAIBLE | 21 - 40 : FAIBLE À MOYEN | 41 - 60 : MOYEN À ÉLEVÉ | 61 - 80 : ÉLEVÉ | 81 - 100 : EXTRÊMEMENT ÉLEVÉ

REMARQUES :

Calcaire argileux de couleur grise.

CONCLUSION ET RECOMMENDATION ÉTAPE 1 :

Selon la norme BNQ 2560-500/2003, un IPPG de **50** est attribué au matériau (potentiel pétrographique de gonflement **moyen à élevé**). Ainsi l'analyse chimique a été réalisé selon la norme BNQ 2560-500/2003.

CALCUL DES MINÉRAUX ÉQUIVALENTS : ÉTAPE 2, BNQ 2560-500/2003

S_{total} (%)	0,83%	Équivalent de pyrite :	1,54%
SO_4 (%)	######	Équivalent de gypse :	
Al_2O_3 (%)		Équivalent de minéraux argileux :	
CO_2 (%)		Équivalent de calcite :	

Intérprétation des résultats de pyrite résiduelle :

0 - 0,5 : NÉGLIGEABLE | 0,5 - 0,75 : FAIBLE | 0,75 - 1,0% : FAIBLE À MOYEN | 1,0 - 1,25 : MOYEN À ÉLEVÉ | 1,25 - 1,5 : ÉLEVÉ | >1,5 % EXTRÊMEMENT ÉLEVÉ

CONCLUSION ET RECOMMENDATION ÉTAPE 2 :

Le pourcentage équivalent en pyrite dans l'échantillon s'élève à **1,54**, ce qui correspond à un potentiel chimique de gonflement **Extrèmement élevé**. En se basant sur les résultats obtenus, nous sommes d'avis que les précautions d'usage (excavation du roc fissuré et/ou oxydé, assèchement, pose de membrane et/ou de béton maigre, etc.) devraient être suivies avant la construction de bâtiment sur ce roc.

PRÉPARÉ PAR : Mustapha El dhimni, géologue **APPROUVÉ PAR :** **DATE :** 2009-10-20

Qualitas

6155, Rue des Tournelles, Québec (Québec) G2J 1P7
Tél. : 418-626-5211 Fax : 418-626-9312 www.tsblic.ca

CALCUL DE L'INDICE PÉTROGRAPHIQUE DU POTENTIEL DE GONFLEMENT (IPPG) BNQ 2560-500

No de l'échantillon : 09-SG-12982
Type de matériau : Roc (carotte)
Provenance : Forage F-09-03, profondeur 1,75 m.
Source première : Forage F-09-03, profondeur 1,75 m.
Usage proposé : Roc au niveau du fond d'excavation
Date de prélèvement : 2009-10-07
Prélevé par : François Blanchet, ing. Jr.
Firme : Groupe Qualitas inc.

Projet : Manège Militaire, Québec, Québec
N. / no de dossier : 09-10-010
V. / no de dossier : -
V. / no d'échantillon : -
Rapport soumis à : Digonnet Stéphane, dir. de projet
Firme : SNC-Lavalin
455, rue René-Lévesque Ouest (Monréal)

FACIÈCES PÉTROGRAPHIQUE	Indice IP	FRACTION GRANULOMÉTRIQUE											
		20 mm et plus *			10/14 mm			5/10 mm			2,5/5 mm		
		g (± 0.1)	% (± 1)	IP x % (± 0,1)	g (± 0.1)	% (± 1)	IP x % (± 0,1)	g (± 0.1)	% (± 1)	IP x % (± 0,1)	g (± 0.1)	% (± 1)	IP x % (± 0,1)
Calcaire argileux laminé	0,50	1000	100	50,0									
Total		1000	100										

* *Roc. L'analyse granulométriue n'est pas requise.*

	20 mm et plus	10/14 mm	5/10 mm	2,5/5 mm
A Sommation IP x % pour chaque fraction granulométrique	50,0	0,0	0,0	0,0
B Pourcentage pondéré de la fraction granulométrique retenue sur le tamis	100			
C Résultat de A x B/100 (Ci)	50,0			

IPPG cumulatif global pondéré (Σ Ci) : 50

Intérprétation des indices pétrographiques de potentiel de gonflement (IPPG) :

0 - 10 : NÉGLIGEABLE | 11 - 20 : FAIBLE | 21 - 40 : FAIBLE À MOYEN | 41 - 60 : MOYEN À ÉLEVÉ | 61 - 80 : ÉLEVÉ | 81 - 100 : EXTRÊMEMENT ÉLEVÉ

REMARQUES :

Calcaire argileux laminé de couleur grise.

CONCLUSION ET RECOMMENDATION ÉTAPE 1 :

Selon la norme BNQ 2560-500/2003, un IPPG de **50** est attribué au matériau (potentiel pétrographique de gonflement **moyen à élevé**). Ainsi l'analyse chimique a été réalisé selon la norme BNQ 2560-500/2003.

CALCUL DES MINÉRAUX ÉQUIVALENTS : ÉTAPE 2, BNQ 2560-500/2003

S_{total} (%)	1,43%	Équivalent de pyrite :	2,67%
SO_4 (%)	######	Équivalent de gypse :	
Al_2O_3 (%)		Équivalent de minéraux argileux :	
CO_2 (%)		Équivalent de calcite :	

Intérprétation des résultats de pyrite résiduelle :

0 - 0,5 : NÉGLIGEABLE | 0,5 - 0,75 : FAIBLE | 0,75 - 1,0% : FAIBLE À MOYEN | 1,0 - 1,25 : MOYEN À ÉLEVÉ | 1,25 - 1,5 : ÉLEVÉ | >1,5 % EXTRÊMEMENT ÉLEVÉ

CONCLUSION ET RECOMMENDATION ÉTAPE 2 :

Le pourcentage équivalent en pyrite dans l'échantillon s'élève à **2,67**, ce qui correspond à un potentiel chimique de gonflement **Extrêmement élevé.** En se basant sur les résultats obtenus, nous sommes d'avis que les précautions d'usage (excavation du roc fissuré et/ou oxydé, assèchement, pose de membrane et/ou de béton maigre, etc.) devraient être suivies avant la construction de bâtiment sur ce roc.

PRÉPARÉ PAR : Mustapha El dhimni, géologue **APPROUVÉ PAR :** **DATE :** 2009-10-20

ii) CHEMICAL ANALYSES

COREM

Maxxam Analytique - Québec

Alain Lemieux
2690, avenue Dalton

Sainte-Foy, Québec
G1P 3S4

RAPPORT D'ANALYSE version 1

COREM

Votre référence ...: A952012

Alain Lemieux
30193 Maxxam Analytique - Québec

Date de réception : 2009-10-15
Certificat émis le : 2009-10-20

Numéro COREM :			28971- 1	28971- 2
Nature :			SOLIDES	SOLIDES
Désignation :			187475-01R/SG-12981	187499-01R/SG-12982
B41-	1	Analyse	2009-10-20	2009-10-20
B41-	1	S total	0.83 %	1.43 %
B76-	1	Analyse	2009-10-20	2009-10-20
B76-	1	SO4	127 mg/kg	179 mg/kg
P02-	1	Préparation	2009-10-15	2009-10-15
P02-	1	Conc.	Terminée	Terminée
P03-	1	Préparation	2009-10-15	2009-10-15
P03-	1	Pulv. P	Terminée	Terminée

Responsable :

Bernard Montminy, chimiste, M.Sc.
Directeur adjoint à la réalisation technique

COREM
1180, rue de la Minéralogie, Québec (Québec) Canada G1N 1X7
Téléphone : (418) 527-8211 Télécopieur : (418) 527-4816

APPENDIX 4

ROCK ANCHOR DESIGN METHOD

1. ROCK ANCHOR DIAGRAM

- L : Total anchor length (m)
- L_s : Bonded length (m)
- L_w : Cone depth (m)
- D : Diameter of the drilled hole (m)
- β : Half angle of the cone apex (°)
- P : Total pullout load (kN)

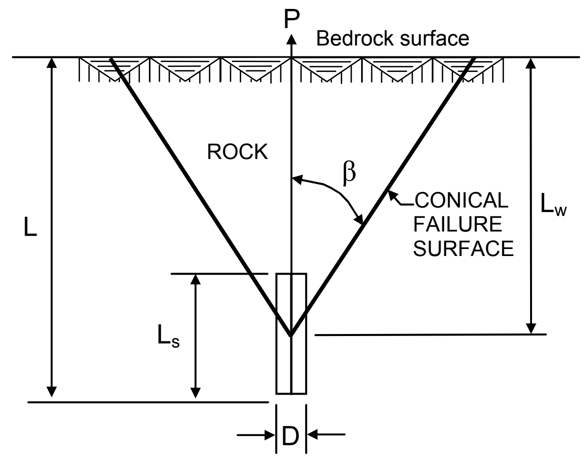


FIGURE 1

2. OBJECTIVES OF THE METHOD

The purpose of an anchorage system is to develop a resistance load higher than the pullout load.

$$R_g \geq P \qquad R_g = R \times \Phi$$

where
- R_g : Factored geotechnical resistance (kN)
- R : Ultimate resistance load (kN)
- P : Total pullout load (kN)
- Φ : Resistance factor

Section 3 calculation below, consider 4 types of failure :

- Tensile stress in the steel rod
- Bond between steel rod and grout
- Bond between rock and grout
- Rock pull-up

The resistance must be calculated for each of these types of failure. The lowest resistance value obtained from those 4 criteria shall be used in the final design.

3. CALCULATION METHOD

TENSILE STRESS IN THE STEEL ROD

The allowable resistance developed by the steel rod in function of the rod characteristics (section, tensile, yield strength …). The steel rod manufacturer will specify the characteristics. The safety factor must be sufficient.

BOND BETWEEN THE STEEL ROD AND THE GROUT

The purpose of this calculation is to obtain a bonded rod length between the steel rod and the grout, which is long enough to develop the allowable resistance. This resistance is determined according to the following formula :

$$R_g = \pi \; d \; L_{s1} \; S_b$$

where d : Rod diameter (m)

L_{s1} : Bonded length between rod and grout (m)

S_b : Bonded strength between rod and grout (kPa)

where $S_b = 0.95 \sqrt{f'_c} \times \Phi \times 1000$ (kPa)

f'_c : Unconfined compression strength of the grout, generally specified as 30 MPa at 28 days (MPa)

Φ : Resistance factor of 0.4

Thus $$L_{s1} = \frac{R_g}{\pi \; d \; S_b}$$

3.3 BOND BETWEEN THE ROCK AND THE GROUT

The purpose of this calculation is to obtain a bonded rod length between the rock and the grout, which is long enough to develop the allowable resistance. This resistance is determined according to the following formula :

$$R_g = \pi \ D \ L_{s2} \ S_r$$

where D : Drilled hole diameter (m)

L_{s2} : Bonded length between rock and grout (m)

S_r : Bonded strength between rock and grout (kPa)

S_r equals the lowest value obtained from the 3 following criteria :

$S_r \leq 0.1 \ q_u \ x \ \Phi$ $\quad$ $S_r \leq 0.1 \ f'_c \ x \ \Phi$ $\quad$ $S_r = 1\,300$ kPa

where q_u : Unconfined compressive strength of the rock (kPa)

f'_c : Unconfined compressive strength of the grout, generally specified as 30 MPa at 28 days (kPa)

Φ : Resistance factor equal to 0.4

Thus $$L_{s2} = \frac{R_g}{\pi \ D \ S_r}$$

Furthermore, the following criteria must also be considered :

a) For fair to excellent rock quality (RQD > 50 %), the bonded length L_{s2} must equal at least 30 times the drilled hole diameter of the anchor.

b) For poor to very poor rock quality (RQD ≤ 50 %), the bonded length L_{s2} must equal at least 40 times the drilled hole diameter of the anchor.

c) For shale or rock with shaly beds, the bonded length L_{s2} must equal at least 80 times the drilled hole diameter of the anchor.

d) For all other cases, the bonded strength L_{s2} must equal at least 3 m.

3.4 ROCK PULL-UP

This calculation is used to evaluate the total anchor length required to ensure that the working load will be resisted safely without failure occurring in the rock mass. For this analysis, it is assumed that for a single rock anchor at failure, an inverted cone of rock is pulled out of the rock mass. The conical failure surface has its apex at the middle of the anchor assuming a contained angle of 2 times β.

$$R_g = L_w^3 \gamma \Phi \tan^2 \beta \qquad L_w = L - \frac{L_s}{2} \quad \text{(see Figure 1)}$$

where
- L_w : Length of the inverted cone, from the middle of the anchor to the bedrock (m)
- L : Total anchor length (m)
- L_s : Bonded length, higher value of L_{s1} and L_{s2} obtained from steps 3.2 and 3.3 (m)
- γ : Unit weight of the rock (kN/m^3)
- β : Half angle of the cone apex (°)
  - β = 30 ° for very poor to poor rock quality (RQD ≤ 50 %)
  - β = 45 ° for fair to excellent rock quality (RQD > 50 %)
- Φ : Resistance factor equal to 0.4

Therefore, the total anchor length is :

$$L = L_w + \frac{L_s}{2}$$

or

$$L = \left(\frac{R_g}{\gamma \Phi \tan^2 \beta}\right)^{1/3} + \frac{L_s}{2}$$

4. INTERACTION OF ANCHORS

4.1 RECOMMENDED EXACT METHOD

For a group of anchors, the interaction of the conical failure surface with that of each adjacent anchor should be taken into account by reducing the load per anchor as followed :

$P' = \psi' P$

where P' : Reduced pullout load due to the interaction of one adjacent anchor (kPa)
P : Pullout load of one single anchor (kPa)
ψ' : Reduction factor to take into account adjacent anchors function of a/r

For 1 adjacent anchor : $\psi' = 0.5 + 0.4\ a/r$ if $0 < a < 1.25\ r$

For 2 adjacent anchors : $\psi' = (0.5 + 0.4\ a/r)^2$ if $0 < a < 1.25\ r$

$\psi' = 1$ if $a \geq 1.25\ r$

where a : Distance between 2 anchors (m)
r : Distance between the center of the anchor and the conical failure surface at the bedrock (m)

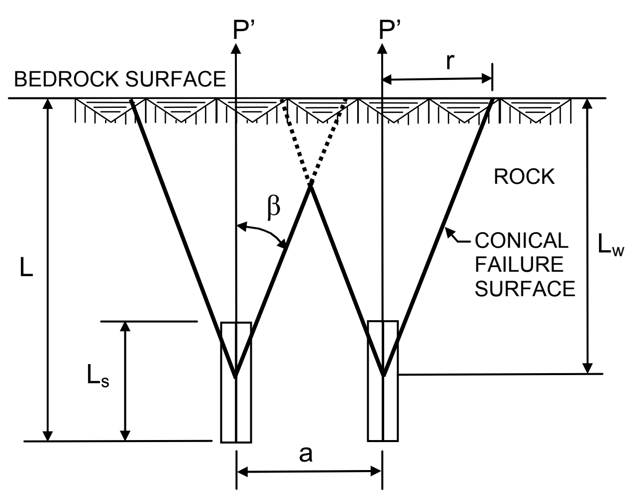


FIGURE 2

4.2 CONCENTRATED ANCHORS, GLOBAL METHOD

A group of closely spaced anchors (between 5 and 10 times the drilled hole diameter) can be considered as one unit in rock pull-up. The rock failure surface forms an inverted truncated pyramid as shown in Figure 3.

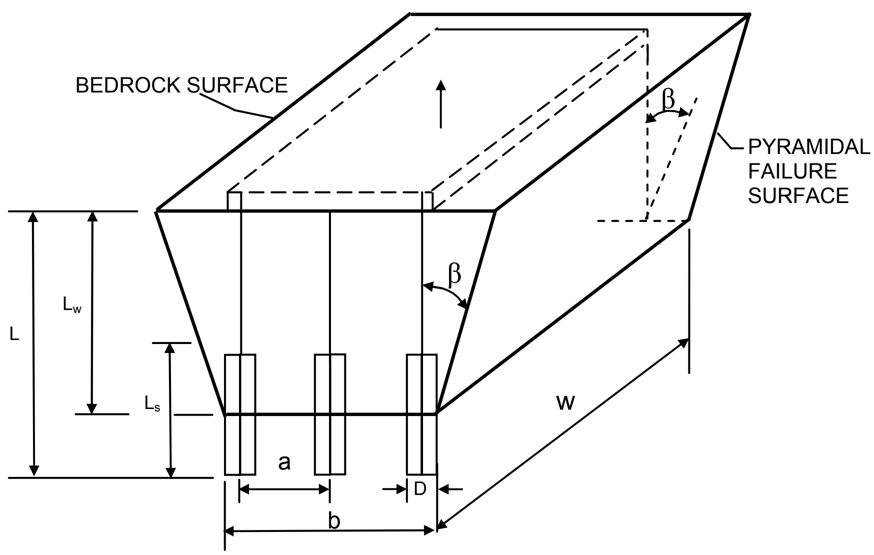


FIGURE 3

For $a < 10\ D$, the resistance load is :

$$R_{gg} = \frac{1}{3}\ \gamma\ \Phi\ L_w\ (A_1 + A_2 + \sqrt{A_1\ A_2})$$

where
- R_{gg} : Global factored geotechnical resistance load (kPa)
- Φ : Resistance factor equal to 0.4
- γ : Unit weight of the rock (kN/m^3)
- L_w : Length of the inverted truncated pyramid from the middle of anchors to the bedrock surface (m)
- A_1 : Area of the group anchors (m^2) ($A_1 = b \times w$)
- A_2 : Area of the upper base of the inverted pyramid (bedrock) (m^2)
  $A_2 = 4\ L_w^2\ \tan^2\beta + 2\ L_w \tan\beta\ (b + w) + b\ w$
- b : Width of the group anchors (m)
- w : Length of the group anchors (m)
- β : Half angle of the cone apex (°)
  - $\beta = 30°$ for very poor to poor rock quality (RQD ≤ 50 %)
  - $\beta = 45°$ for fair to excellent rock quality (RQD > 50 %)
- a : Distance between 2 anchors (m)
- D : Diameter of the drilled hole (m)

5. FURTHER RECOMMENDATIONS

The minimal distance between 2 adjacent anchors must be greater than 5 times the diameter of the drilled hole in the rock.

The holes have to be filled up with a lean grout above the bonded length in order to protect the anchors.

Two anchors will have to be tested on the site. The maximum load must reach at least 1.33 times the calculated resistance load R_g.

6. REFERENCES

1) BUREAU SECURITAS. *Recommandations concernant la conception, le calcul, l'exécution et le contrôle des tirants d'ancrage,* Eyrolles Editions, Paris, 1972.

2) LITTLEJOHN, G.S. and D.A. Bruce. *Rock Anchors – State of the Art – Part 1: Design, Ground Engineerin*g, May 1975, Vol. 8, n° 3.

3) RADHAKRISHNA, H.S., J.J. Deans and F. Devisser. *Shallow Rock Anchors,* The Canadian Geotechnical Society, Papers for a Symposium on Anchor Systems in Geotechnical Engineering, 1986.

4) NAVAL FACILITIES ENGINEERING COMMAND. *Design Manual – Soil Mechanics, Foundations and Earth Structures,* Virginia, 1971.

APPENDIX 5

REPORT SCOPE

REPORT SCOPE

1. **USE OF REPORT**

A. Project modifications: The factual data, interpretations and recommendations contained in this report refer to the specific project described in the report, and do not apply to any other project or site. Should the project be modified from a design, dimension, location or level point of view, Qualitas Inc. will have to be consulted so that we can confirm that the recommendations previously made remain valid and applicable.
B. Number of boreholes: The recommendations made in this report are only intended to serve as a guide to the design engineer. The number of boreholes needed to determine all underground conditions that can affect construction (costs, techniques, equipment, schedule, etc.) should normally be higher than the number needed for dimensioning. Contractors who bid or subcontract the work should rely on their own studies and their own interpretations of borehole factual results to form an appreciation of how the underground conditions could affect their work.

2. **DRILLING REPORTS AND INTERPRETATION OF UNDERGROUND CONDITIONS**

A. Soil and rock descriptions: The soil and rock descriptions in this report are based on commonly accepted classification and identification methods used in geotechnical practice. Soil and rock classification and identification call for judgment. Such descriptions can differ from those that another geotechnician with similar knowledge of good geotechnical practices might give.
B. Soil and rock conditions at borehole locations: Drilling reports only provide subsurface conditions at the borehole locations. The boundaries between the various strata on the drilling reports are often approximate, corresponding instead to transition zones, and were thus subject to interpretation. The accuracy with which underground conditions are indicated depends on the drilling method, sampling method and frequency, and uniformity of the terrain encountered. Borehole spacing, sampling frequency and type of drilling are also dictated by budget and schedule considerations beyond the control of Qualitas Inc.
C. Soil and rock conditions between boreholes: Soil and rock formations vary over a more or less greater extent. Underground conditions between boreholes may vary with respect to the conditions encountered in the boreholes. Any interpretation of conditions between boreholes involves some risk. Such interpretations may lead to the discovery of conditions that differ from those anticipated. Qualitas Inc. cannot be held liable for the discovery of soil and rock conditions different from those described elsewhere than in the places where the boreholes were drilled.
D. Groundwater levels: The groundwater levels given in this report correspond solely to those observed in the place and date indicated in the report. These conditions may vary seasonally or as the result of construction on the site or on adjacent sites. Such variations are beyond the control of Qualitas Inc.

3. **STUDY AND CONSTRUCTION FOLLOW UP**

A. Final phase verification: Not all design and construction details are known at the time this report is issued. We therefore recommend that the services of Qualitas Inc. be retained to shed light on the consequences construction may have on the finished structure.
B. Inspection during execution: We recommend that the services of Qualitas Inc. be retained during construction to verify and confirm that subsurface conditions over the entire extent of the site do not differ from those given in the report, and that construction work will not have any negative impact on site conditions.

4. **CHANGED CONDITIONS**: The soil conditions described in this report are those observed at the time of the study. Unless otherwise indicated, these conditions form the basis of the report recommendations. Soil conditions can be altered significantly by construction work (traffic, excavation, etc.) on the site or on adjacent sites. An excavation can expose soil to changes due to humidity, drying or frost. Unless otherwise indicated, the soil should be protected against such changes or reworking during construction.

   When the conditions encountered on the site differ significantly from those provided in this report due to the heterogeneous nature of the subsoil or construction work, it is up to the client and user of this report to notify Qualitas Inc. of any changes and to provide Qualitas Inc. with an opportunity to review the recommendations in this report. Recognizing changes in soil conditions requires a certain amount of experience. We therefore recommend that an experienced geotechnical engineer be seconded to the site to verify whether conditions have undergone any significant changes.

5. **DRAINAGE**: Groundwater drainage is often required for temporary as well as permanent project installations. Improper drainage design or execution can have serious consequences. Qualitas Inc. can in no case assume responsibility for the effects of drainage unless Qualitas Inc. is specifically involved in the detailed design and construction supervision of the drainage system.

6. **ENVIRONMENTAL CONDITIONS**: In some cases, land on which Qualitas Inc. carries out its investigations may have been subject to contaminant spills, or the water table may contain pollutants originating from a site outside the land under study. Such conditions require an environmental characterisation study. The present geotechnical study was not carried out based on such a study. It should be noted that environmental laws and regulations can have significant impacts on project viability, orientation and costs. Such laws and regulations are subject to amendment and will have to be verified and taken into account during the project design and preparation phase.

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