GEOTECHNICAL INVESTIGATION REPORT, LL987 BADGELEY ISLAND LIGHTHOUSE, Killarney, ON

File: 121622847

Prepared for:

Canadian Coast Guard,
Central and Arctic Region.
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August 7, 2019

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1.0 INTRODUCTION

Stantec Consulting Ltd. (Stantec), acting at the request of Bailey Humphries, of the Canadian Coast Guard (CCG), has completed a geotechnical investigation at the site of an existing lighthouse located along the southeast shore of Badgeley Island located in the northwest potion of Georgian Bay approximately 11 km southwest of the community of Killarney, ON. The purpose of the geotechnical investigation was to obtain information on the soil, bedrock and groundwater conditions to support design of a new aluminum frame lighthouse structure. All work was conducted according to the Stantec proposal dated May 2, 2019, File No. 727026.

Limitations associated with this report and its contents are provided in the Statement of General Conditions included in **Appendix A**.

2.0 SITE AND PROJECT INFORMATION

The location of the proposed lighthouse structure is along the southeast shoreline of Badgeley Island located in the northern portion of Georgian Bay at coordinates 45º55’41.97”N; 81º36’7.56” W. The existing lighthouse is located on the shoreline of Badgeley Island approximately 20 m from the waters edge. The ground surface is relatively flat, consisting of bare limestone bedrock, with frequent rock slabs and boulders exposed at the ground surface.

The general location of the site is shown on Drawing No. 1 in **Appendix B**.

It is our understanding that the existing metal frame lighthouse is to be replaced with a 28 foot tall steel Claymar Tower with a diameter of 1500 mm, as outlined on the Generic Structural Design Plans provided by the Client (Drawing No. S201 to S209 inclusive dated 2006/04/05, prepared by Gloss Associates Inc.).

3.0 SCOPE OF WORK

The scope of work for this geotechnical investigation included the following:

- A field investigation consisting of 1 borehole within 5 m of the base of the existing lighthouse to characterize the soil, bedrock, and groundwater conditions at the location of the proposed Claymar Tower Lighthouse;
- Laboratory testing consisting of grain size analysis, Atterberg limits testing, unconfined compressive strength tests on selected rock core samples (if encountered) and soil corrosivity testing;
- Documentation of the results of the field investigation and laboratory results in a report; and
- Geotechnical input on site preparation and lighthouse foundation design within the report.

4.0 GEOTECHNICAL INVESTIGATION

The field drilling program for the geotechnical investigation was carried out on July 25, 2019. The approximate borehole location is shown on Drawing No. 1 in **Appendix B**.

4.1 INVESTIGATIVE PROCEDURES

As a component of our field procedures, Stantec hired a barge service operated by Herberts Fisheries of Killarney, ON. Herbert Fisheries provided proof of commercial vessel registration and operator certification as per the Canadian Shipping Act.

The boreholes were drilled with a portable rock coring equipment NQ size core barrel (50 mm) supplied and operated by Stata Drilling Group of Sudbury, ON. The subsurface stratigraphy encountered in the boreholes was recorded in the field by experienced Stantec personnel and while logging the rock cores in accordance to ASTM D2113. For each run the Total Core Recovery (TCR), Solid Core Recovery (SCR) and Rock Quality Designation (RQD) was recorded and the cores were placed in wooden core boxes, labelled and returned to our laboratory for visual inspection logging and laboratory analysis.

As bedrock was encountered at the ground surface there were no soil samples generated during the field program.

4.2 LABORATORY TESTING

As there were no soil samples generated during the drilling program there was no laboratory testing such as grain size analysis, Atterberg limits testing and soil corrosivity completed.

Select portions of the rock core were selected for compressive strength testing at our Markham ON, geotechnical laboratory. The results of the compressive strength tests are discussed in the text of this report.

4.3 SURVEY

The elevations at the borehole location was referenced to the existing ground surface elevation.

5.0 SUBSURFACE PROFILE

Soil classification was based on the procedures described in ASTM D2113. A detailed log of the soil conditions encountered is provided on the borehole record BH01, in **Appendix C**.

A brief summary of the stratigraphy is provided in Table 5.1.

Table 5.1 Subsurface Profile

Layer Thickness (m)	Soil Description
1.5 m	Weathered Limestone
1.5 m	Fresh Limestone
1.2 m	Fair Limestone

5.1 LIMESTONE BEDROCK

The limestone bedrock was exposed at the ground surface at the borehole location and surrounding the existing lighthouse tower. The bedrock is described as limestone with shale partings and was weathered to a depth of 1.5 m.

The bedrock consisting of grey, highly weathered to fresh limestone was encountered in the borehole which extended to the maximum investigated depth of 4.2 m. Occasional clay infilling was observed within the fractures. The fractures contained black or rust-brown (iron) staining. The Total Core Recovery (TCR) of the bedrock core ranged from 92% to 100%. The Solid Core Recovery (SCR) of the bedrock core varied from 90% to 96%. The Rock Quality Determination (RQD) of the bedrock core runs ranged from 22.5 % to 81%. The low value was encountered in the upper 1.5 m where the limestone was observed to be slightly to moderately weathered. Consistent with the methods described in the Canadian Foundation Engineering Manual (2006 Edition), the range in RQD indicates that the bedrock should be classified as being of poor to good quality.

Two core samples were selected for unconfined compressive strength testing. The results are summarized in Table 5.2.

Table 5.2 Unconfined Compressive Strength of Bedrock Core

Borehole	Sample Depth (m)	Unconfined Compressive Strength (MPa)	Rock Type
BH 01	2.1 – 2.5	29.3	Limestone
BH -01	3.5 – 3.7	94.1	Limestone

Based on the results of the unconfined compressive strength tests the limestone bedrock is medium strong to strong.

5.2 GROUNDWATER CONDITIONS

Groundwater was not observed in the open borehole on completion of drilling; however, the water level of Georgian Bay was approximately 1.2 m to 1.5 m lower than the borehole elevation and should be assumed to represent the groundwater level on site. Due to the proximity to Georgian Bay the groundwater levels will fluctuate seasonally and in response to lake levels. As such, groundwater conditions encountered during construction may differ from those observed during the geotechnical investigation.

6.0 DISCUSSION AND RECOMMENDATIONS

Based on the information provided by the Client, it is understood that the existing lighthouse is to be removed and replaced with a Steel Claymore Tower structure approximately 8.5 m (28 feet) m in height. Details of the proposed steel structure were provided in Structural Design Drawings S201 to 209 inclusive prepared by Glos Associates Inc. and dated 2006/04/05. The drawings indicate that the tower will consist of a steel frame structure supported on a concrete foundation.

The following recommendations are based on several assumptions outlined throughout this report and the site conditions encountered at the time of the investigation.

The following general development considerations and constraints are provided with respect to the subsurface conditions encountered and the intended scope of development:

- Bedrock was encountered at the ground surface which indicates that excavation of the underlying bedrock will be required for the lighthouse foundations The extent of bedrock excavation would be in the order of 1.5 m to achieve the intended embedment depth into the bedrock.
- Excavation of the bedrock will require use of hydraulic rock breaking equipment (hoe-ramming) or blasting (if permitted).
- The use of conventional shallow foundation types placed on limestone bedrock would be suitable to support the lighthouse structure.
- A gravity base concrete pad or footings anchored into bedrock would be feasible for the planned lighthouse structure.

6.1 SEISMIC SITE CLASSIFICATION

The site classification was determined based on confirmation of bedrock at ground surface at the proposed lighthouse locations.

Based on the conditions observed, a Site Class B should be used for seismic loads and effect in accordance with Table 4.1.8.4.A of the National Building Code of Canada (2015), for foundations bearing on rock.

A copy of the NBC Seismic Hazard Calculation Data sheet is provided in **Appendix E** for reference.

6.2 LIGHTHOUSE FOUNDATIONS

As noted, the conditions at the site consists of limestone bedrock at the ground surface. The upper 1.5 m of the limestone was characterized as being slightly to moderately weathered and of poor quality, with good to fair quality limestone extending from 1.5 m to a depth of 4.2 m.

The design drawings noted above call for 35 mm diameter anchor bolts 1524 mm long extending into the concrete foundations at each of the 8 base plate locations. In this regard the concrete foundations will need to extend a minimum of 1.5 m below the ground surface or the concrete base will need to extend above the ground surface elevation to accommodate the anchor bolts.

6.2.1 Geotechnical Resistance

Foundations bearing on rock may be designed using a factored geotechnical resistance of 1000 kPa (1 MPa), SLS bearing resistance will not govern the design. The factored geotechnical resistance at ULS incorporates a resistance factor of 0.6. If a dead-man anchor, or massive block, is used, that depends upon passive soil resistance against lateral loading, the soil within the frost penetration depth 1.5 m should not be relied upon.

Table 6.1 provides the geotechnical resistance factors to be used for design, in accordance with CSA Group S37-13 for Antennas, towers, and antenna-supporting towers.

Table 6.1 Geotechnical Resistance Factors – Anchored Foundation

Condition	Geotechnical Resistance Factor
Pull-out / uplift for anchors in rock	0.50
Lateral Resistance for anchors in rock	0.75

6.2.2 Uplift Resistance

The uplift capacity of the foundations will be principally calculated based on the unit weight of the foundation using the calculation method presented in **Appendix E** titled “Calculation of Uplift Resistance of Spread Footings”.

6.2.3 Coefficient of Sliding Friction

Table 6.2 summarizes the coefficients of friction between concrete and bedrock, estimated in accordance with the Canadian Foundation Engineering Manual (2006).

Table 6.2 Unfactored Friction Coefficient for Sliding Resistance

Condition	Unfactored Friction Coefficient
Between concrete and bedrock	0.7

6.2.4 Rock Anchor Design

Due to the shallow depth of rock an anchored foundation is recommended to provide adequate resistance for uplift and overturning. For the design of rock anchors, the following design parameters may be considered for the rock mass:

- An unfactored (ultimate) rock to grout bond stress of 1000 kPa may be used for holes grouted with non-shrink grout having a minimum compressive strength of 30 MPa.

- The minimum fixed anchor length (i.e., the length over which the rock to grout bond stress is developed) should be no less than 3 m.
- The unbonded length of anchor should be equal to the height of the rock cone minus half of the bonded length.
- Load testing of the anchors should be carried out to confirm the capacity of the anchors has been achieved.
- A 60° (apex angle) failure cone with the apex located at the midpoint of the bonded length as shown on the sheet titled “Rock Anchor Resistance to Rock Mass Failure” in **Appendix E** should be used for design.

6.3 EXCAVATIONS AND BACKFILL

6.3.1 Excavations

Subsequent to undertaking the site preparation activities as described above, excavation on the Site should be practical using large size, tracked excavation equipment, rock breakers and rippers.

Temporary excavations for the construction of footings must be carried out in accordance with the latest edition of the Occupational Health and Safety Act (OHSA).

Where excavations extend into the bedrock the side slopes can be cut to near vertical. All loose rock must be removed from the rock face and from the upper edge of the excavation.

6.3.2 Backfilling

6.3.2.1 **Foundation Wall Backfill**

Backfill adjacent to the perimeter foundation walls should consist of free-draining, non-frost susceptible granular materials, such as OPSS Granular ‘B’ or low strength concrete.

The backfill should be compacted to a minimum of 98% of the Standard Proctor Maximum Dry Density, with due care to avoid damage to the foundations.

6.4 CONSTRUCTION DEWATERING

A requirement for dewatering (removal of subsurface water in advance of excavation for construction) will be dependent on the depth of excavation and volume of groundwater infiltration anticipated from fractures and seems in the limestone bedrock.

Groundwater level is anticipated to be at or near the level of Georgian Bay which was approximately 1.5 m lower than the borehole elevation. The groundwater levels are anticipated to fluctuate with levels of Georgian Bay.

For reference, the current MOECP regulations state that an Environmental Activity and Sector Registry (EASR) is required for dewatering of between 50,000 L/day and 400,000 L/day and a Permit to Take Water (PTTW) is required for dewatering in excess of 400,000 L/day.

6.5 ADVERSE WEATHER CONDITIONS

Additional precautions, effort, and measures may be required, when and where construction is undertaken during late fall, winter, and/or early spring (i.e., when temperatures and climatic conditions can have an adverse influence on the standard construction practices) or during periods of inclement weather. With respect to all earthworks activities undertaken during the late fall through late spring, when less-than-ideal construction conditions may prevail, the following comments are provided:

1. All of the Structural Fill should consist of OPSS Granular A or B (type II) materials. The use of non-granular fill materials may be considered but obtaining suitable compaction of these materials would be problematic.
2. Fill placement should be inspected by qualified geotechnical personnel on a full-time basis, with the authority to stop the placement of fill at any time when conditions are considered to be less than favourable.
3. Imported materials that contain ice, snow, or any frozen material should not be accepted for use.
4. Overnight frost penetration may occur, even in granular fill materials, where precipitation and ground surface runoff pools and accumulated, and freezing temperatures exist. Any frozen materials should be removed prior to placing subsequent lifts of Structural Fill. Breaking the frost in-situ is not considered acceptable.
5. It may be necessary to stop the placement of Structural Fill during periods of cold, where ambient temperatures are -5°C or less, exist.
6. Concrete should not be placed over frozen subgrade. Once concrete is placed the subgrade must be protected from freezing.

The placement of Structural Fill materials, grout, and concrete, during cold weather conditions requires extra effort beyond that typical when better climatic conditions prevail. At any time where conditions are deemed unfavourable, the placement operation may need to be suspended.

Additional considerations for heating of concrete, heating of forms and reinforcing steel, protection of concrete from freezing, and similar measures may also be required subject to climatic conditions at the time of construction.

7.0 CLOSING COMMENTS

The recommendations made in this report are in accordance with our present understanding of the project and assumptions as outlined throughout this report. Continued geotechnical engineering involvement during the project should be maintained to ensure the recommendations as outlined in this report are adhered to.

Use of this report is subject to the Statement of General Conditions provided in Appendix A. It is the responsibility of Canadian Coast Guard, who is identified as “the Client” within the Statement of General Conditions, and its agents, to review the conditions and to notify Stantec Consulting Ltd. should any of these not be satisfied. The Statement of General Conditions addresses the following:

- Use of the report;
- Basis of the report;
- Standard of care;
- Interpretation of site conditions;
- Varying or unexpected site conditions; and
- Planning, design or construction.

This report has been prepared by Peter Healy, C.E.T. and reviewed by Ron Howieson, P.Eng.

We appreciate the opportunity to complete this work, if we can be of further assistance please contact the undersigned at your convenience.

Yours very truly,

STANTEC CONSULTING LTD.

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Appendix A

A.1 STATEMENT OF GENERAL CONDITIONS

STATEMENT OF GENERAL CONDITIONS

USE OF THIS REPORT: This report has been prepared for the sole benefit of the Client or its agent and may not be used by any third party without the express written consent of Stantec Consulting Ltd. and the Client. Any use which a third party makes of this report is the responsibility of such third party.

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

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

INTERPRETATION OF SITE CONDITIONS: Soil, rock, or other material descriptions, and statements regarding their condition, made in this report are based on site conditions encountered by Stantec Consulting Ltd. at the time of the work and at the specific testing and/or sampling locations. Classifications and statements of condition have been made in accordance with normally accepted practices which are judgmental in nature; no specific description should be considered exact, but rather reflective of the anticipated material behavior. Extrapolation of in situ conditions can only be made to some limited extent beyond the sampling or test points. The extent depends on variability of the soil, rock and groundwater conditions as influenced by geological processes, construction activity, and site use.

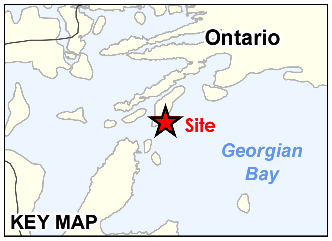
VARYING OR UNEXPECTED CONDITIONS: Should any site or subsurface conditions be encountered that are different from those described in this report or encountered at the test locations, Stantec Consulting Ltd. must be notified immediately to assess if the varying or unexpected conditions are substantial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

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

Appendix B

B.1 DRAWINGS

T:\Autocad\Drawings\Project Drawings\2019\CAD2019\121622847\121622847_Borehole Plan.mxd   Revised: 2019-08-07 By: gbriones



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Stantec

Project Location
Badgely Island
Georgian Bay

Project No. 121622847
Prepared by GBB
on 2019-08-07

Client/Project
CANADIAN COAST GUARD
BADGELY ISLAND LIGHTHOUSE

Drawing No.
1

Title
Borehole Location Plan

Notes
1. Coordinate System: NAD 1983 UTM Zone 17N.
2. Imagery: Source: Esri, DigitalGlobe, GeoEye, Earthstar Geographics, CNES/Airbus DS, USDA, USGS, AeroGRID, IGN, and the GIS User Community.

Appendix C

C.1 SYMBOLS AND TERMS USED ON BOREHOLE AND TEST PIT RECORDS

C.2 BOREHOLE RECORDS

SYMBOLS AND TERMS USED ON BOREHOLE AND TEST PIT RECORDS

SOIL DESCRIPTION

Terminology describing common soil genesis:

Rootmat	- vegetation, roots and moss with organic matter and topsoil typically forming a mattress at the ground surface
Topsoil	- mixture of soil and humus capable of supporting vegetative growth
Peat	- mixture of visible and invisible fragments of decayed organic matter
Till	- unstratified glacial deposit which may range from clay to boulders
Fill	- material below the surface identified as placed by humans (excluding buried services)

Terminology describing soil structure:

Desiccated	- having visible signs of weathering by oxidization of clay minerals, shrinkage cracks, etc.
Fissured	- having cracks, and hence a blocky structure
Varved	- composed of regular alternating layers of silt and clay
Stratified	- composed of alternating successions of different soil types, e.g. silt and sand
Layer	- > 75 mm in thickness
Seam	- 2 mm to 75 mm in thickness
Parting	- < 2 mm in thickness

Terminology describing soil types:
The classification of soil types are made on the basis of grain size and plasticity in accordance with the Unified Soil Classification System (USCS) (ASTM D 2487 or D 2488) which excludes particles larger than 75 mm. For particles larger than 75 mm, and for defining percent clay fraction in hydrometer results, definitions proposed by Canadian Foundation Engineering Manual, 4th Edition are used. The USCS provides a group symbol (e.g. SM) and group name (e.g. silty sand) for identification.

Terminology describing cobbles, boulders, and non-matrix materials (organic matter or debris):
Terminology describing materials outside the USCS, (e.g. particles larger than 75 mm, visible organic matter, and construction debris) is based upon the proportion of these materials present:

Trace, or occasional	Less than 10%
Some	10-20%
Frequent	> 20%

Terminology describing compactness of cohesionless soils:
The standard terminology to describe cohesionless soils includes compactness (formerly "relative density"), as determined by the Standard Penetration Test (SPT) N-Value - also known as N-Index. The SPT N-Value is described further on page 3. A relationship between compactness condition and N-Value is shown in the following table.

Compactness Condition	SPT N-Value
Very Loose	<4
Loose	4-10
Compact	10-30
Dense	30-50
Very Dense	>50

Terminology describing consistency of cohesive soils:
The standard terminology to describe cohesive soils includes the consistency, which is based on undrained shear strength as measured by *in situ* vane tests, penetrometer tests, or unconfined compression tests. Consistency may be crudely estimated from SPT N-Value based on the correlation shown in the following table (Terzaghi and Peck, 1967). The correlation to SPT N-Value is used with caution as it is only very approximate.

Consistency	Undrained Shear Strength		Approximate SPT N-Value
	kips/sq.ft.	kPa	
Very Soft	<0.25	<12.5	<2
Soft	0.25 - 0.5	12.5 - 25	2-4
Firm	0.5 - 1.0	25 - 50	4-8
Stiff	1.0 - 2.0	50 – 100	8-15
Very Stiff	2.0 - 4.0	100 - 200	15-30
Hard	>4.0	>200	>30

ROCK DESCRIPTION

Except where specified below, terminology for describing rock is as defined by the International Society for Rock Mechanics (ISRM) 2007 publication "The Complete ISRM Suggested Methods for Rock Characterization, Testing and Monitoring: 1974-2006"

Terminology describing rock quality:

RQD	Rock Mass Quality
0-25	*Very Poor Quality*
25-50	*Poor Quality*
50-75	*Fair Quality*
75-90	*Good Quality*
90-100	*Excellent Quality*

Alternate (Colloquial) Rock Mass Quality	
Very Severely Fractured	*Crushed*
Severely Fractured	*Shattered or Very Blocky*
Fractured	*Blocky*
Moderately Jointed	*Sound*
Intact	*Very Sound*

RQD (Rock Quality Designation) denotes the percentage of intact and sound rock retrieved from a borehole of any orientation. All pieces of intact and sound rock core equal to or greater than 100 mm (4 in.) long are summed and divided by the total length of the core run. RQD is determined in accordance with ASTM D6032.

SCR (Solid Core Recovery) denotes the percentage of solid core (cylindrical) retrieved from a borehole of any orientation. All pieces of solid (cylindrical) core are summed and divided by the total length of the core run (It excludes all portions of core pieces that are not fully cylindrical as well as crushed or rubble zones).

Fracture Index (FI) is defined as the number of naturally occurring fractures within a given length of core. The Fracture Index is reported as a simple count of natural occurring fractures.

Terminology describing rock with respect to discontinuity and bedding spacing:

Spacing (mm)	Discontinuities	Bedding
>6000	*Extremely Wide*	-
2000-6000	*Very Wide*	*Very Thick*
600-2000	*Wide*	*Thick*
200-600	*Moderate*	*Medium*
60-200	*Close*	*Thin*
20-60	*Very Close*	*Very Thin*
<20	*Extremely Close*	*Laminated*
<6	-	*Thinly Laminated*

Terminology describing rock strength:

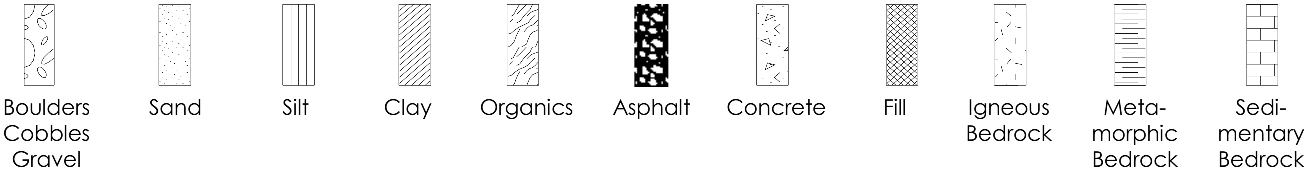
Strength Classification	Grade	Unconfined Compressive Strength (MPa)
Extremely Weak	R0	<1
Very Weak	R1	1 – 5
Weak	R2	5 – 25
Medium Strong	R3	25 – 50
Strong	R4	50 – 100
Very Strong	R5	100 – 250
Extremely Strong	R6	>250

Terminology describing rock weathering:

Term	Symbol	Description
Fresh	W1	No visible signs of rock weathering. Slight discoloration along major discontinuities
Slightly	W2	Discoloration indicates weathering of rock on discontinuity surfaces. All the rock material may be discolored.
Moderately	W3	Less than half the rock is decomposed and/or disintegrated into soil.
Highly	W4	More than half the rock is decomposed and/or disintegrated into soil.
Completely	W5	All the rock material is decomposed and/or disintegrated into soil. The original mass structure is still largely intact.
Residual Soil	W6	All the rock converted to soil. Structure and fabric destroyed.

STRATA PLOT

Strata plots symbolize the soil or bedrock description. They are combinations of the following basic symbols. The dimensions within the strata symbols are not indicative of the particle size, layer thickness, etc.



SAMPLE TYPE

SS	Split spoon sample (obtained by performing the Standard Penetration Test)
ST	Shelby tube or thin wall tube
DP	Direct-Push sample (small diameter tube sampler hydraulically advanced)
PS	Piston sample
BS	Bulk sample
HQ, NQ, BQ, etc.	Rock core samples obtained with the use of standard size diamond coring bits.

WATER LEVEL MEASUREMENT

▼	measured in standpipe, piezometer, or well
▽	inferred

RECOVERY

For soil samples, the recovery is recorded as the length of the soil sample recovered. For rock core, recovery is defined as the total cumulative length of all core recovered in the core barrel divided by the length drilled and is recorded as a percentage on a per run basis.

N-VALUE

Numbers in this column are the field results of the Standard Penetration Test: the number of blows of a 140 pound (63.5 kg) hammer falling 30 inches (760 mm), required to drive a 2 inch (50.8 mm) O.D. split spoon sampler one foot (300 mm) into the soil. In accordance with ASTM D1586, the N-Value equals the sum of the number of blows (N) required to drive the sampler over the interval of 6 to 18 in. (150 to 450 mm). However, when a 24 in. (610 mm) sampler is used, the number of blows (N) required to drive the sampler over the interval of 12 to 24 in. (300 to 610 mm) may be reported if this value is lower. For split spoon samples where insufficient penetration was achieved and N-Values cannot be presented, the number of blows are reported over sampler penetration in millimetres (e.g. 50/75). Some design methods make use of N-values corrected for various factors such as overburden pressure, energy ratio, borehole diameter, etc. No corrections have been applied to the N-values presented on the log.

DYNAMIC CONE PENETRATION TEST (DCPT)

Dynamic cone penetration tests are performed using a standard 60 degree apex cone connected to 'A' size drill rods with the same standard fall height and weight as the Standard Penetration Test. The DCPT value is the number of blows of the hammer required to drive the cone one foot (300 mm) into the soil. The DCPT is used as a probe to assess soil variability.

OTHER TESTS

S	Sieve analysis
H	Hydrometer analysis
k	Laboratory permeability
γ	Unit weight
G_s	Specific gravity of soil particles
CD	Consolidated drained triaxial
CU	Consolidated undrained triaxial with pore pressure measurements
UU	Unconsolidated undrained triaxial
DS	Direct Shear
C	Consolidation
Q_U	Unconfined compression
I_p	Point Load Index (I_p on Borehole Record equals $I_p(50)$ in which the index is corrected to a reference diameter of 50 mm)

	Single packer permeability test; test interval from depth shown to bottom of borehole
	Double packer permeability test; test interval as indicated
	Falling head permeability test using casing
	Falling head permeability test using well point or piezometer

1 of 1

BOREHOLE RECORD

BH1

CLIENT Canadian Coast Guard

LOCATION

DATES: BORING July 25, 2019 WATER LEVEL -

BOREHOLE No. BH1

PROJECT No. 121622847

DATUM

DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	STRATA PLOT	WATER LEVEL	SAMPLES TYPE	NUMBER	RECOVERY (mm)	N-VALUE OR RQD
0		Poor to good quality, grey LIMESTONE with shale laminations			NQ	1	98%	23%
1								
2					NQ	2	100%	81%
3								
4					NQ	3	92%	52%
		End of Borehole						
5								
6								
7								
8								
9								
10								

UNDRAINED SHEAR STRENGTH - kPa

50 100 150 200

WATER CONTENT & ATTERBERG LIMITS W_P W W_L

DYNAMIC PENETRATION TEST, BLOWS/0.3m ★

STANDARD PENETRATION TEST, BLOWS/0.3m ●

10 20 30 40 50 60 70 80 90

$\underline{\nabla}$ Inferred Groundwater Level

$\underline{\blacktriangledown}$ Groundwater Level Measured in Standpipe

▣ Field Vane Test, kPa

☐ Remoulded Vane Test, kPa

△ Pocket Penetrometer Test, kPa

Appendix D

D.1 LABORATORY ANALYSIS

Stantec

ROCK CORE COMPRESSIVE STRENGTH

Client: Canadian Coast Guard
Project: Badgeley Island Lighthouse
Material Description: Limestone Bedrock
Date Tested: Aug.06,2019

Project No.: **121622847**
Lab No.: **246**
Tested By: **Bahram Siavash**

BH1	**RC2**	**7'-7'6"**
		Average
LENGTH (mm)	98.34	97.7
	97.27	
	97.48	
DIAMETER (mm)	48.64	48.6
	48.6	
	48.61	
L/D	2.01	
Area m^2	0.0018554	
WEIGHT (kg)	0.478	
Volume (m^3)	0.0001813	
Unit Weight (kg/m^3)	2637	
LOAD	(lb)	12209
	N	54308.1
	MPa	**29.3**

BH1	**RC1**	**11'6"-12'0"**
LENGTH (mm)	95.39	95.4
	95.5	
	95.42	
DIAMETER (mm)	48.96	48.7
	48.58	
	48.64	
L/D	1.96	
Area m^2	0.0018638	
WEIGHT (kg)	0.493	
Volume (m^3)	0.0001779	
Unit Weight (kg/m^3)	2772	
LOAD	(lb)	39413
	N	175316.9
	MPa	**94.1**

		Average
LENGTH (mm)		#DIV/0!
DIAMETER (mm)		#DIV/0!
L/D	#DIV/0!	
Area m^2	#DIV/0!	
WEIGHT (g)		
Volume (m3)	#DIV/0!	
Unit Weight (kg/m3)	#DIV/0!	
LOAD	(lb)	
	N	0.0
	MPa	**#DIV/0!**

Appendix E

E.1 ROCK CORE PHOTOS

Figure 1: Rock Core – Badgeley Island

Appendix F

F.1 CALCULATION OF UPLIFT RESISTANCE FOR SPREAD FOOTINGS AND ROCK ANCHOR RESISTANCE TO ROCK MASS FAILURE

Calculation of Uplift Resistance of Spread Footings

The allowable uplift resistance of spread footings may be determined from the submerged unit weight of the soil block located above the footing, the dead weight of the footing and an appropriate factor of safety (typically = 2). The soil block used in the calculation of the uplift resistance is defined by imaginary lines drawn at 20° angles upward and away from the top edges of the footing, as per the diagram below:

$$U = (W + W_C) / F \text{ and } U \geq P$$

where: U = **Allowable uplift resistance (kN)**

P = **Vertical uplift force (kN)**

F = **Factor of safety equal to 2**

W = **Weight of soil block above the footing (kN)**

$$W = \tfrac{1}{3} D \gamma' \left(A_1 + A_2 + \sqrt{(A_1 A_2)}\right)$$

W_C = **Weight of concrete footing (kN)**

$$W_C = B \times L \times T \times \gamma'_C$$

γ' = **Submerged unit weight of backfill soil (kN/m^3)**

A_1 = **Area of footing footprint (m^2)**

$$A_1 = B \times L$$

A_2 = **Area of topside of the soil block (m^2)**

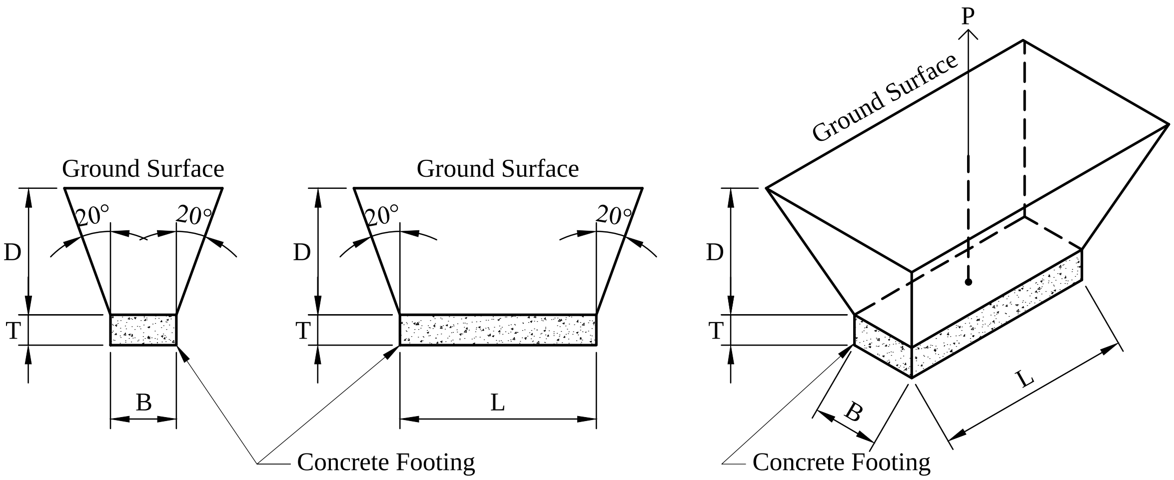
$$A_2 = 0.5\,D^2 + 0.7\,D(B+L) + B \times L$$

B = **Width of concrete footing (m)**

L = **Length of concrete footing (m)**

T = **Thickness of concrete footing (m)**

γ'_C = **Submerged unit weight of concrete (= 13.7 kN/m^3)**



Rock Anchor

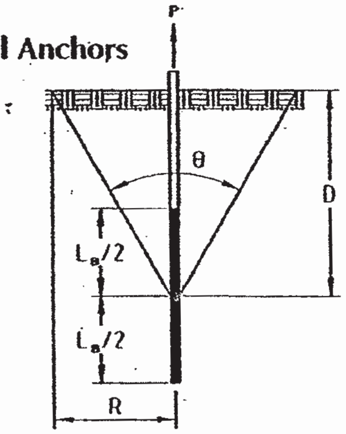
Resistance to Rock Mass Failure

Required Safety Factor for Resistance to Rock Mass Failure: $W_R / P \geq 2.0$

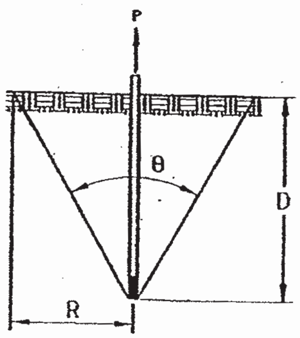
Design Considerations:

1. **Use 60° or 90° apex angle as per recommendations in the geotechnical report**

Vertical Anchors

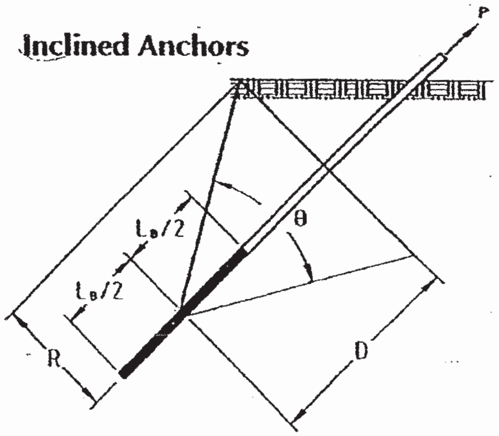


Grouted Rock Anchors

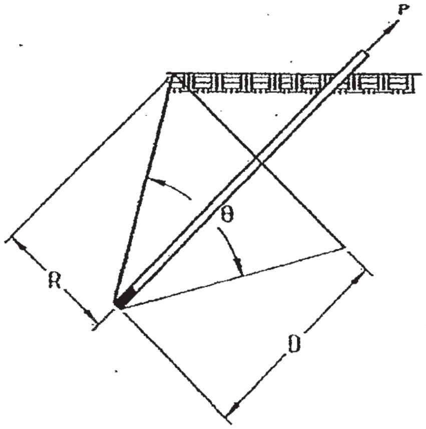


Mechanical Rock Anchors

Inclined Anchors



Grouted Rock Anchors



Mechanical Rock Anchors

P = Resultant of maximum axial anchor forces
D = Height of rock cone
R = Radius of rock cone
θ = Appex angle
L_B = Bond length
γ_R = Submerged unit weight of bedrock
W_R = Weight of rock cone ($W_R = \frac{1}{3}\pi R^2 D \gamma_R$)

Appendix G

G.1 SEISMIC HAZARD CALCULATION

2015 National Building Code Seismic Hazard Calculation

INFORMATION: Eastern Canada English (613) 995-5548 français (613) 995-0600 Facsimile (613) 992-8836
Western Canada English (250) 363-6500 Facsimile (250) 363-6565

Site: 45.928N 81.602W **User File Reference:** Badgeley Island Lighthouse

2019-08-07 13:12 UT

Requested by: Stantec Consulting Ltd.

Probability of exceedance per annum	0.000404	0.001	0.0021	0.01
Probability of exceedance in 50 years	2 %	5 %	10 %	40 %
Sa (0.05)	0.059	0.035	0.023	0.007
Sa (0.1)	0.084	0.052	0.034	0.012
Sa (0.2)	0.085	0.055	0.037	0.014
Sa (0.3)	0.075	0.049	0.034	0.013
Sa (0.5)	0.063	0.042	0.029	0.010
Sa (1.0)	0.040	0.026	0.017	0.005
Sa (2.0)	0.021	0.013	0.008	0.002
Sa (5.0)	0.005	0.003	0.002	0.001
Sa (10.0)	0.002	0.001	0.001	0.000
PGA (g)	0.048	0.030	0.020	0.007
PGV (m/s)	0.051	0.032	0.020	0.006

Notes: Spectral (Sa(T), where T is the period in seconds) and peak ground acceleration (PGA) values are given in units of g (9.81 m/s^2). Peak ground velocity is given in m/s. Values are for "firm ground" (NBCC2015 Site Class C, average shear wave velocity 450 m/s). NBCC2015 and CSAS6-14 values are highlighted in yellow. Three additional periods are provided - their use is discussed in the NBCC2015 Commentary. Only 2 significant figures are to be used. **These values have been interpolated from a 10-km-spaced grid of points. Depending on the gradient of the nearby points, values at this location calculated directly from the hazard program may vary. More than 95 percent of interpolated values are within 2 percent of the directly calculated values.**

References

National Building Code of Canada 2015 NRCC no. 56190; Appendix C: Table C-3, Seismic Design Data for Selected Locations in Canada

Structural Commentaries (User's Guide - NBC 2015: Part 4 of Division B)
Commentary J: Design for Seismic Effects

Geological Survey of Canada Open File 7893 Fifth Generation Seismic Hazard Model for Canada: Grid values of mean hazard to be used with the 2015 National Building Code of Canada

See the websites www.EarthquakesCanada.ca and www.nationalcodes.ca for more information