TULLOCH

M E M O R A N D U M

Date: Thursday, November 17, 2022

To: Ken Becking, P.Eng.
Director of Public Works
Township of Muskoka Lakes
1 Bailey St., P.O. Box 129
Port Carling, ON P0B 1J0

From: Erik Giles P.Eng., Kelvin Cheung E.I.T.

CC: George Liang P.Eng.

RE: Burgess Dam – North Slope Geotechnical Investigation and Slope Stability Analysis

Dear Mr. Becking,

TULLOCH was retained by The Township of Muskoka Lakes (The Client) to perform a site investigation adjacent to the North Slope downstream of the Burgess 1 Generating Station Powerhouse in Bala, Ontario. The scope of work included the advancement of three (3) sampled boreholes on River Street adjacent to the Burgess 1 Generating Station. The purpose of the investigation was to further understand the subsurface soil and shallow bedrock conditions of the area to aid in development of mitigation or rehabilitation options for the slope. Drawing 20-1051-G-01 attached to this memorandum presents a site plan detailing borehole location for the geotechnical investigation completed for this project.

The memorandum will discuss a brief overview of the regional local geology, summary of the investigation methodology and factual findings, followed by a description of the analysis undertaken, and presentation of rehabilitation options. Terminology as it pertains to the borehole logs and memorandum is attached. Detailed borehole logs including individual soil layers and descriptions are also attached to this document, as well as analysis results.

1. INTRODUCTION AND SCOPE

The slope directly north of the Burgess 1 Generating Station is located downstream of the dam and directly downstream of the powerhouse. An existing concrete retaining wall, approximately 7.25 m long, keys into the north side of the powerhouse. Gabion baskets provide support below the retaining wall and extend approximately 11 m beyond the retaining wall limits in the downstream direction. At the toe of the gabions, there appears to be historically placed or dumped rock fill that varies in height and size. Generally, the restricted slope areas near the powerhouse are overgrown, while the sloped area downstream is grass covered.

The scope of work for this memorandum as part of the larger Burgess Rehabilitation Project is outlined below, it includes:

- Geotechnical Site Investigation (including Borehole Drilling, Soil Sampling and Description, etc.)
- Detailed Description of factual subsurface conditions including laboratory testing and standard geotechnical testing
- Slope Stability Analysis including development of preliminary mitigation and rehabilitation options for the North Slope identified above
- Delivery of one (1) Engineering Geotechnical Memorandum for detailing the findings of the analysis and the preliminary options for remediation/rehabilitation of the North Slope based on the soil properties and in-situ groundwater measurements. The recommendations in this memo will be input into the overall preliminary design of the rehab of the Burgess 1 Dam facility.

It is noted that two (2) boreholes were originally proposed on the South side of River St., with one (1) proposed on the north side. Due to hazards associated with overhead powerlines on the South side of River St., all three (3) boreholes were advanced on the north side of River St.

2. REGIONAL GEOLOGY

Based on review of Bedrock Geology and Surficial Geology of Southern Ontario mapping as published by the Ontario Geological Society (OGS), the site surficial geology is comprised of Canadian Shield with formations of Precambrian Bedrock typical within the Muskoka region. The typical geologic formations for the Bala area including hard and smooth pink to grey migmatitic rocks as well as quartzofeldspathic gneisses (OGS 2019). The Burgess 1 Dam is located at the lower section of the Muskoka River watershed near the bottom of Lake Muskoka where regional topography is typically mapped as low local relief varying from plains to undulating hummocky conditions. Overburden in the Bala area is typically sandy and shallow in depth with thick organic deposits found in low lying wetland areas.

3. SITE INVESTIGATION AND METHODOLOGY

The geotechnical investigation program included the following scope of work:

1. Borehole investigations on September 9th, 2020, including three (3) sampled boreholes in total, labelled BH-20-01 to BH-20-03.

2. Bedrock coring was completed in BH-20-01. Core logging of all rock core samples retrieved during the investigation was completed during the execution of the borehole. Cores were logged immediately upon retrieval, and measurements for Rock Quality Designation (RQD) were obtained to determine bedrock quality.

Drawing 20-1051-G-01 attached presents a site plan detailing borehole locations for the geotechnical investigation.

3.1 Geotechnical Borehole Summary

A summary of the boreholes drilled on the site are shown below in Table 3-1.

Table 3-1: Summary of Borehole Information

Borehole No.	Elevation[1] (m)	Northing[1] (m)	Easting[1] (m)	Bedrock Depth[2] (mbgs)	Borehole Depth[2] (mbgs)
BH-20-01	225.1	609067	4985600	1.47	4.5
BH-20-02	224.7	609059	4985601	1.24[3]	1.2
BH-20-03	224.4	509053	4985601	1.78[3]	1.8

Note(s):[1] Elevation and Borehole Coordinates are shown in UTM 17T Datum. [2] Meters below ground surface (mbgs), rounded to nearest 0.1 m. [3] Inferred bedrock depth.

Boreholes were advanced using a CME55 truck-mounted drill rig owned and operated by Landcore Drilling from Chelmsford, Ontario. The boreholes were advanced using hollow stem augers. Bedrock cores were retrieved within the NW casing via diamond rotary with an NQ2 (76 mm OD) rock core barrel. The rig was equipped with standard soil sampling equipment including an automatic hammer.

During the geotechnical drilling, soil samples were obtained using standard split spoon equipment in conjunction with Standard Penetration Tests (SPT) conducted in accordance with ASTM D1586 procedures. SPT sampling generally occurred at semi continuous 0.76 m intervals. In the bedrock, core samples were generally retrieved in 1.5 m continuous runs with an NQ2 core barrel. The bedrock was logged in the field and Rock Quality Designation (RQD) was calculated on site as the core runs were retrieved.

The drilling and soil sampling programs were directed by a TULLOCH representative, who logged the drilling operations and identified the soil samples as they were retrieved. The recovered soil and rock cores were transported to TULLOCH's CCIL Certified Laboratory in Sault Ste. Marie, ON. Detailed borehole logs are attached to this memorandum.

4. LABORATORY TESTING PROGRAM

A geotechnical laboratory testing program was performed on representative soil and rock core samples in accordance with ASTM standards. Table 4-1 provides a list of the testing program. Detailed laboratory reports for the particle size analysis and unconfined compressive strength of rock tests, can be found attached to this memorandum.

Table 4-1: Summary of Rock Laboratory Testing Program

Test	Number of Tests	ASTM Standards
Particle Size Analysis	2	ASTM D422
Unconfined Compressive Strength (Rock)	2	ASTM D7012

5. SUMMARY OF SUBSURFACE CONDITIONS

5.1 General

The following section outlines the soil deposits/stratigraphy and corresponding depths encountered during the investigation. Further details can be found in the attached borehole logs.

It should be noted that the soil boundaries indicated on the borehole logs are inferred from non-continuous sampling and observations during drilling. These boundaries are intended to reflect approximate transition zones for the purpose of geotechnical design and should not be interpreted as exact planes of geological change. Further, in boreholes where bedrock coring was not undertaken, depths to bedrock are inferred based on auger refusal.

5.2 Stratigraphy Overview

A total of three (3) boreholes were advanced to assess the subsurface conditions on River St. and the adjacent North Slope. All boreholes were advanced to refusal, BH-20-01 was cored to confirm and assess the shallow bedrock conditions. Throughout the boreholes, 125 mm of asphalt was found to overly road base fills consisting of gravelly sand to sand some gravel. In BH-20-01 auger grinding occurred from below the asphalt to bedrock surface at 1.47 m, inferred to be caused by the presence of cobbles and boulders. Bedrock was confirmed at 1.47 m in BH-20-01 and was inferred at 1.2 and 1.8 mbgs in BH-20-02 and -03 respectively. In BH-20-01, bedrock was found to be granitic gneiss, fine to medium grained with angled foliation. The rock was slightly weathered to fresh, and strong with unconfined compression strengths ranging from 100.3 MPa in Run 1 to 130.3 MPa in Run 2.

A simplified stratigraphic profile, and bedrock depths for each borehole is summarized below in Table 5-1. Further details with individual soil layers and characteristics can be viewed in the detailed borehole logs attached to this memorandum.

Table 5-1: Summary of Soil and Bedrock Conditions

Borehole No.	Ground Surface Elevation[1] (m)	Investigation Profile (mbgs)	Bedrock Depth (mbgs)[2]	Bedrock RQD Range (%)
BH-20-01	225.1	0.00-0.13, Asphalt 0.13-1.47, (SW) Sand, some gravel	1.47	56-94
BH-20-02	224.7	0.00-0.13 Asphalt 0.13-1.24, (SW) Sand, some gravel	1.24[3]	-
BH-20-03	224.4	0.00-0.13 Asphalt 0.13-1.78, (SW) Sand, some gravel	1.78[3]	-

Note(s):[1] Elevation and Borehole Coordinates are shown in UTM 17T Datum. [2] Meters below ground surface (mbgs). [3] Inferred bedrock depth.

5.3 Groundwater Conditions

Groundwater was measured upon completion of each borehole location. A summary of groundwater measurements taken in the boreholes is presented in Table 5-2 below. Groundwater readings were taken down hole upon drilling completion, as such the ground water levels measured on site may not represent static conditions.

Table 5-2: Water Level Readings Summary

Borehole No.	Surface Elevation (m)	Groundwater Depth[1] (mbgs)
BH-20-01	225.1	4.12
BH-20-02	224.7	Not encountered
BH-20-03	224.4	Not encountered

Note(s):[1] Meters below ground surface (mbgs)

Groundwater level is subject to seasonal fluctuations with high levels occurring during wet weather conditions in the spring and fall and lower levels during dry weather conditions.

6. NORTH SLOPE STABILITY ANALYSIS

The following sections will discuss the results of the stability modelling of the existing North Slope retaining wall, gabion basket wall and the overall global slope stability. The modelling was based on review of available drawings, topographic survey, and the encountered stratigraphy from the geotechnical investigation.

6.1 Retaining Wall and Gabion Stability Analysis

Concrete retaining wall global stability and gabion wall global and internal stability calculations were conducted for the North Slope area. Using the data collected from the geotechnical investigation, and topographic survey the initial Factor of Safety (FOS) calculations were completed to help frame the recommendations in the following sections. The FOS calculation for stability analysis of the gabion and retaining wall sections are based on the following Equations:

FOS against sliding failure:

$$FOS = \frac{\sum Resisting\ Froce}{\sum Driving\ Force} \quad [1\text{-}1]$$

FOS against overturning failure:

$$FOS = \frac{\sum Resisting\ Moment}{\sum Driving\ Moment} \quad [1\text{-}2]$$

Table 6-1 summarizes the geotechnical parameters used in the stability calculations. Geotechnical parameters were based on the results of the geotechnical investigation and TULLOCH's engineering experience for conservative design purposes.

Table 6-1: Summary of Geotechnical Parameters Stability Calculation[1]

No.	Type of Material	Cohesion, c' (kPa)	Internal Friction Angle,φ' (Degree)	Unit Weight, γ' (kN/m³)
1	Silty Sand Fill	0	35	19
2	Rockfill	0	38	20
3	Gabion Basket	30	38	20
4	Retaining Wall Concrete	-	-	24
5	Concrete to Rock Interface	-	38	-

Note(s): [1]-Geotechnical parameters are assumed based on TULLOCH's engineering experience.

6.1.1 Gabion Stability Results

Geometry used in stability analysis of the gabion retaining wall was based on the available historical information and observations during site inspection. For global stability, the external boundary of the gabion retaining wall structure was taken to be from the toe of the gabion basket (Gabion 1) retaining wall to the upstream edge of the upper most gabion basket (Gabion 4). The gabion wall is assumed to be founded on bedrock as no construction records or design drawings were available for the structure. Gabion basket widths are all taken to be 1m for the purposes of the stability calculation based on review of available historical drawings. Active and passive earth pressure coefficients have been modified to consider the sloping backfill geometry of the North

Slope above the gabion wall. Table 6-2 summarizes the required and calculated factors of safety for the stability of the gabion basket retaining wall.

Table 6-2: Calculated FOS for Stability of Gabion Basket Retaining Wall

Stability Case	Stability Case	FOS	Minimum Required FOS
Global	Sliding	1.69	1.5
	Overturning	7.64	2.0
Gabion 1	Sliding	**1.05**	1.5
Gabion 2	Sliding	**1.40**	1.5
Gabion 3	Sliding	2.15	1.5
Gabion 4	Sliding	5.08	1.5

It should be noted that based on the available survey data, traffic loading on top of the slope is within the active wedge zone and therefore is applied to the gabion wall calculations. This is a preliminary assessment with limited investigation data and the geometry of gabion wall inferred from the inspection.

Based on the above results, the stability of the gabion basket retaining wall is in a marginally unsafe condition. The internal stability of the wall does not meet the required safety factor with respect to sliding. The rockfill at the toe of the wall has been ignored in this analysis due to its discontinuous nature, however, in reality it may provide minor support to the lower two gabions. Continued deterioration and movement of the wall will likely cause further instability if left unchecked. Therefore, action is recommended to remediate or replace the Gabion Wall which will be discussed in Section 7.

6.1.2 Existing Concrete Retaining Wall Stability Results

Geometry used in stability analysis of the concrete retaining wall was based on the available historical information and provided drawings as well as observation during site inspection. Based on the historical drawings, the concrete retaining wall is assumed to be founded on bedrock. Table 6-3 summarizes the required and calculated factors of safety for the stability of the retaining wall. A sensitivity analysis was conducted based on the U/S water level of the retaining wall as a subdrain for the wall was not presented in the drawing nor established during the site inspection of the wall. As such in a flooding event similar to 2019 water could build up behind the wall causing additional force on the wall.

Table 6-3: Calculated FOS for Stability of Concrete Retaining Wall

Stability Case	Stability Case	FOS	Minimum Required FOS
U/S water level at surface of U/S fill	Sliding	2.5	1.5
	Overturning	**1.7**	2
U/S water level 0.5 m below surface of U/S fill	Sliding	3.0	1.5
	Overturning	2.2	2

It should be noted that based on the available survey data, the traffic loading is within the active wedge zone of the backfill and therefore is applied to the concrete retaining wall calculations. This is a preliminary assessment with limited investigation data and the geometry of concrete wall is inferred from the inspection and available historical information.

Based on the results, the existing concrete retaining wall is typically in a safe condition. However, when the U/S water level is high, i.e., at the surface of the fill, the factor of safety decreases to a marginally safe condition with the required Safety Factor for overturning not being met. This condition likely occurs during period of high precipitation, during the spring freshet and is also likely during an overtopping event. Buildup of water pressure on the upstream side of the wall is expected due to the lack of drains through the retaining wall. It is also noted that a large, open vertical crack exists in the retaining wall which indicates historic movement. Continued deterioration and movement of the wall may cause further reduction in overall stability if left unchecked.

6.2 North Slope Global Stability Analysis

Limit equilibrium global stability analysis was conducted for the North Slope area using Geostudio 2021 R2, version 11.1.3.22700 by GEOSLOPE International Ltd. Survey data collected as part of the 2019 DSR for the Burgess Dam, information from the geotechnical investigation, and limited available historical information, was used to generate analysis geometry and determine a critical section which is shown in Figure 6-1 Below. It should be noted that the bedrock profile in the model is assumed based on local site and regional geology characteristics. The phreatic surface was assumed based on typical powerhouse tailwater elevation and the groundwater conditions encountered during the geotechnical investigation. See Figure 6-1 below.

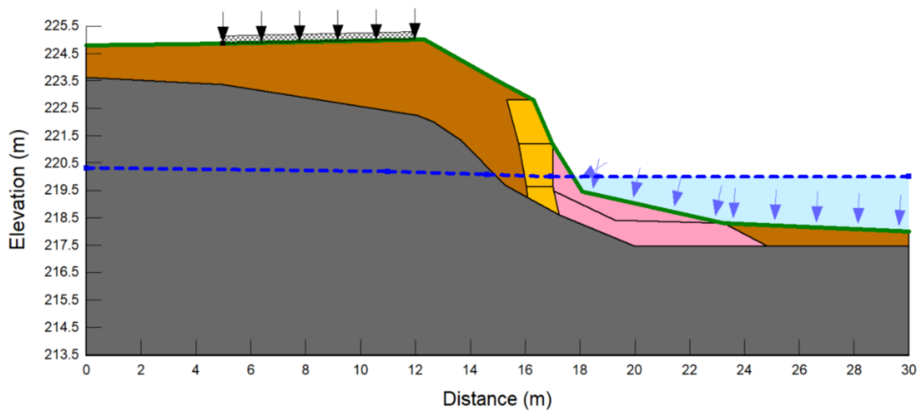


Figure 6-1: Slope Stability Geometry and Phreatic Surface

The slope stability model resulted in a global factor of safety of 1.24, the required factor of safety for the current site conditions is typically 1.5. A sensitivity study where the gabion basket netting has deteriorated was also run, this yielded a factor of safety of 0.61 showing that without a gabion wall in good condition, the slope is unsafe and would likely fail. The condition of the gabion wall below the rockfill at the downstream toe is unknown as it is covered in rock fill, however given its age and the fair condition of the existing gabion wall it is reasonable to assume that the gabions are nearing the end of their service life and it is recommended that they be rehabilitated or replaced.

7. ENGINEERING DISCUSSION

The following section will discuss engineering recommendations for the North Slope and associated structures to be incorporated into the preliminary design of the Burgess 1 Generating Station facility. The Gabion Basket Existing Retaining Wall and overall North Slope will be discussed.

The existing concrete retaining wall is noted to have extended vertical cracks from the crest to the soil contact on the downstream side. Further, typical features of modern retaining walls including subdrain system, and reinforcement in the form of anchor points or dowelling were not apparent on historical drawings or observed during the last DSR conducted in 2019. This indicates that the wall is in fair condition and should be rehabilitated or replaced. Given the planned rehabilitation of the overall facility replacement or remediation of this wall is recommended at this time.

The gabion wall is noted to be in marginally unsafe condition, with some unknowns as to the geometry and foundation. The North Slope is noted to be steep at approximately a 1.75 to 1 (H:V).

The various North Slope stability analyses indicate that the concrete retaining wall, gabion wall and north slope areas are all in a marginally safe condition. Given the above information, the following remediation options are presented for consideration.

7.1 Option 1 – Remediation of Existing Concrete Retaining and Gabion Basket Walls

With the various components of the North Slope area in fair to poor condition, remediation of the existing structure should be considered. This would include remediation of the existing concrete retaining wall and reinforcement and possible replacement of the existing Gabion Wall.

The following recommendations should be implemented for rehabilitation of the North Slope area:

- Subdrains should be installed in the concrete retaining wall to prevent pore pressure buildup on the upstream side, drains should be run into the tailrace area to prevent additional erosion. Surface run-off should be collected and diverted away from the retaining wall section.
- Cracks in the concrete retaining wall should be repaired and if required additional structural reinforcement should be added.
- Anchoring of the concrete retaining wall into the shallow bedrock should be considered to improve stability in overturning and sliding.
- The concrete retaining wall and repair locations should be regularly inspected for further movement over time. A monitoring system could be implemented on the wall to track movement in the future.
- Removal of rockfill at the toe of the gabion wall to inspect the lower Gabions and determine their condition, the Gabions could then be remediated or replaced as required. Adequately sized rip rap and/or larger gabion stone could be used to prevent erosion and help stabilize the North Slope.
- The North Slope should be monitored regularly for signs of instability or movement.

Rehabilitation may extend the service life of the walls and the North Slope; however, it would require regular monitoring and maintenance with potential for eventual replacement as the structures in question are aging and near the end of their service life.

7.2 Option 2 – Replacement of Concrete and Gabion Basket Retaining Walls

With future plans for upgrades to the current Burgess Dam structures including dam raising, powerhouse rehabilitation and improvements to the tailrace, this presents a good opportunity to replace the existing North Slope retaining structures and incorporate a more robust retention system for River Street. Though construction of properly engineered retaining structures requires larger initial investment, it will have reduced maintenance costs, increased safety of the walls and surrounding infrastructure, and minimized risk to power generation in the long term. Given the required rehabilitation of the Generating Station and Dam it may be difficult to replace these North Slope infrastructure at a later point which could increase cost when eventual replacement is required. The following recommendations should be implemented in North Slope area.

- Removal of existing concrete and gabion basket retaining walls.
- Removal of existing fill and native materials to competent bedrock.
- Construction of a concrete training wall dowelled into bedrock and tied into the Powerhouse, extending to the current downstream limit of the gabion wall. The concrete training wall should include subdrains.
- Construction of a replacement concrete retaining wall tied into the powerhouse and founded on bedrock, which should include subdrains.
- Backfilling behind and between all structures should be an approved free draining granular fill such as OPSS Granular B Type II or equivalent backfill compacted to 98% of the Standard Proctor Maximum Dry Density (SPMDD). Placed in compacted lifts of maximum loose lift thickness of 300 mm.
- Regrading of all slopes above the gabion wall to 2:1 (H:V) or less.

Extending a training wall from the powerhouse will prevent erosion of the North Slope and allow for significantly better control of water through the powerhouse particularly during high flow events. Furthermore, the heightened and improved training wall will act as a retaining wall for the North Slope and provide better structural resistance to the North Slope allowing the infrastructure to perform better and mitigate the risks associated with slope failure on the site.

A preliminary drawing will be issued for the training wall as part of the preliminary design memo for the Burgess 1 Generating Station. It should be noted that the recommendations in the memorandum are preliminary in nature. It is recommended that the calculations and remediation options be re-evaluated in the detailed design phase to ensure that they meet the needs of the Township.

8. CLOSURE

This geotechnical memorandum has been prepared by TULLOCH for the exclusive use of the Client and their authorized agents. Within the limitations of scope, schedule and budget, our services have been executed in accordance with generally accepted practices in the field of geotechnical engineering, for the above noted location. Classification and identification of soils, and geologic units have been based upon commonly accepted methods employed in professional geotechnical practice. No warranty or other conditions, expressed or implied, should be understood. Please refer to the Notice to Reader attached, which is an integral part of this report.

We trust that the information in this report will be sufficient to allow the Client to proceed with the project. Should further elaboration be required for any portion of this project, we would be pleased to assist.

Sincerely,

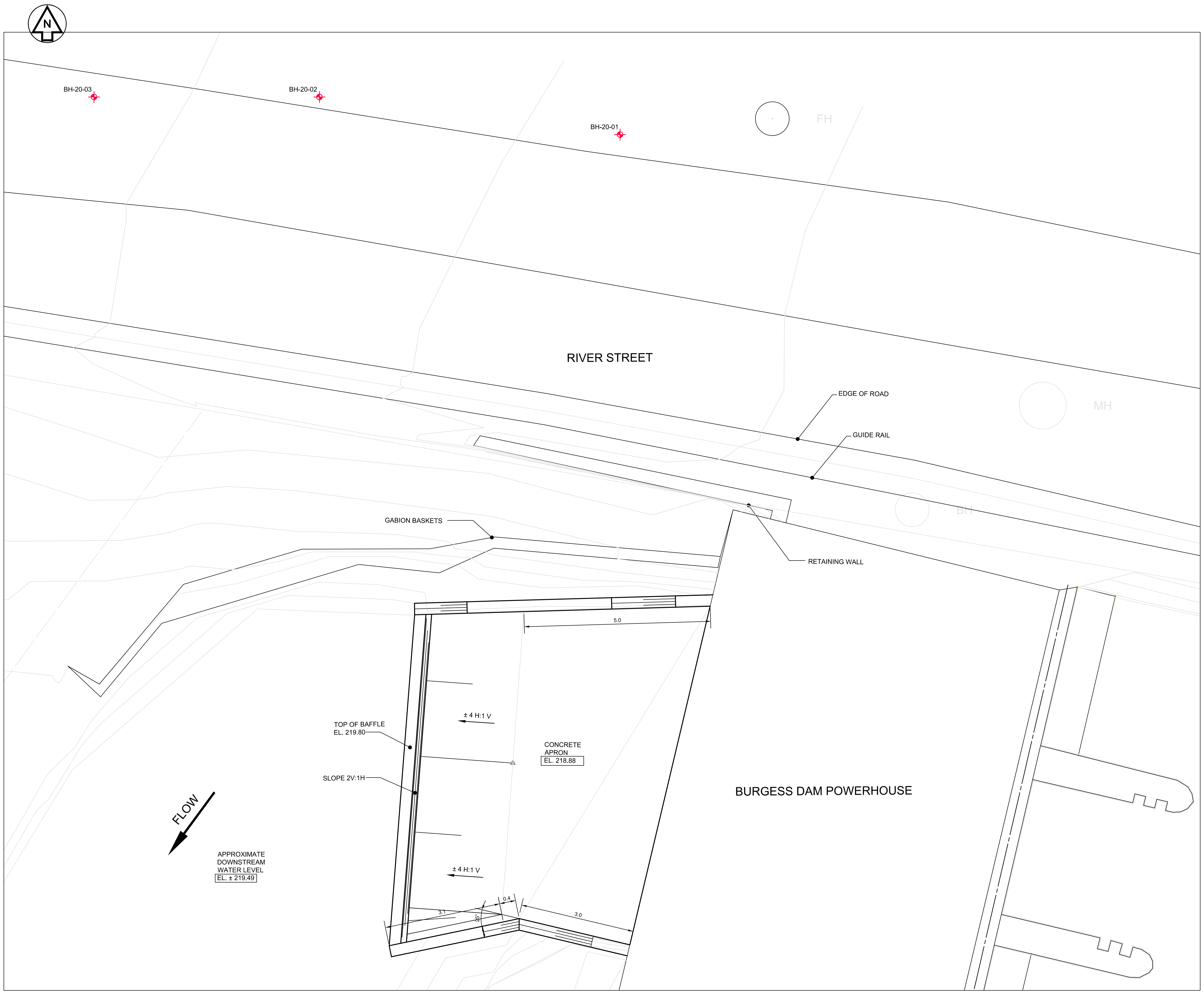
Kelvin Cheung B.Sc. E.I.T
Engineer in Training

Reviewed By:
Erik Giles P.Eng.
Geotechnical Engineer

LICENSED PROFESSIONAL ENGINEER
E. K. GILES
100206670
NOV 17/2022
PROVINCE OF ONTARIO

Attachment(s): Site Plan, Terminology, Site Photo Log, Borehole Logs, Rock Core Photos, Laboratory Data, Slope Stability Results, Notice to Reader

Site Plan



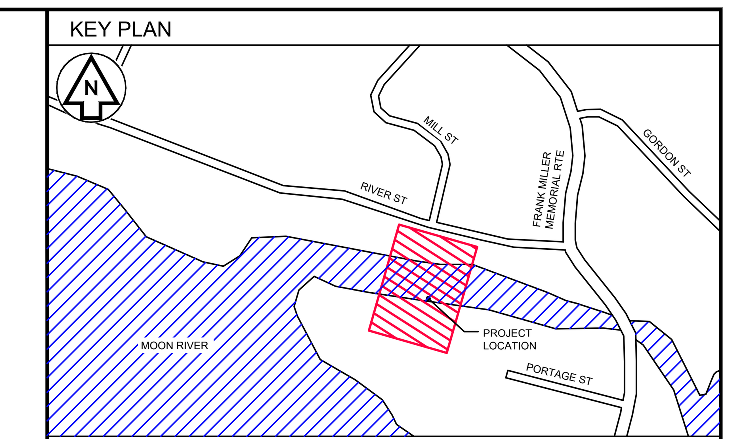
PLAN

SCALE: 1:50

0 0.5 1 1.5 2 2.5 m

Scale: 1:50

KEY PLAN



LEGEND:

BH-20-01 — BOREHOLE LOCATION

NOTES:

1. CO-ORDINATES ARE IN UTM ZONE 17 (NAD83 CSRS).

BOREHOLE LOCATIONS

BOREHOLES	EASTING	NORTHING	ELEVATION
BH-20-01	609 067	4 985 600	225.1
BH-20-02	609 059	4 985 601	224.7
BH-20-03	609 053	4 985 601	224.4

No.	DATE	BY	DESCRIPTION
A	2022-02-23	KK	ISSUED FOR INTERNAL REVIEW

REVISIONS

TULLOCH

CLIENT:
TOWNSHIP OF MUSKOKA LAKES

PROJECT:
LITTLE BURGESS GENERATING STATION REHAB

DRAWING:
NORTH EMBANKMENT GEOTECHNICAL INVESTIGATION PLAN

DRAWN BY:	CHECKED BY:	DESIGNED BY:
M. ANYS	E GILES	K KORTEKAAS
APPROVED BY:	SCALE:	DATE:
C STILLWELL	AS NOTED	2022-02-23
DRAWING No.: 20-1051-G-01		REVISION No.: A

Terminology

ABBREVIATIONS, TERMINOLOGY AND PRINCIPAL SYMBOLS USED IN REPORT AND BOREHOLE LOGS

BOREHOLES AND TEST PIT LOGS

Soils

AA	Auger Sample	w	Water Content
SS	Split Spoon	wP	Plastic Limit
TO	Tin-walled Tube	wL	Liquid Limit
TP	Thin-walled Piston	V(FV)	Field Vane
WS	Washed Sample	OR	Organic Content
SC	Soil Core	GR	Gravel
BS	Block Sample	SA	Sand
WH	Weight of rods & hammer	SI	Silt
WR	Weight of rods	CL	Clay

Bedrock

TCR	Total Core Recover	VN	Vein
SCR	Solid Core Recovery	CO	Contact
FI	Fracture frequency index	KV	Karstic void
HQ	Rock Core (63.5 mm dia.)	MB	Mechanical Break
NQ	Rock Core (47.6 mm dia.)	PL	Planar
BQ	Rock Core (36.5 mm dia.)	CU	Curved
JN	Joint	UN	Undulating
FLT	Fault	IR	Irregular
SH	Shear	SM	Smooth
K	Slikensided	SR	Slightly Rough
BD	Bedding	R	Rough
FO	Foliation	VR	Very rough

IN SITU SOIL TESTING

Standard Penetration Test (SPT) "N" value. The number of blows required to drive a 51 mm OD split barrel sampler into the soil a distance of 300 mm with a 63.5kg weight free falling a distance of 760 mm after an initial penetration of 150 mm has been achieved.

Dynamic Cone Penetration Test (DCPT) is the number of blows required to drive a cone with a 60 degree apex attached to "A" size drill rods continuously into the soil for each 300 mm penetration with a 63.5 kg weight free falling a distance of 760 mm.

Cone Penetration Test (CPT) is an electronic cone point with a 10 cm base area with a 60 degree apex pushed through the soil at a penetration rate of 2cm/s.

Field Vane Test (FVT) consists of a vane blade, a set of rods and torque measuring apparatus used to determine the undrained shear strength of cohesive soils.

SOIL DESCRIPTIONS

The soil descriptions and classifications are based on an expanded Unified Soil Classification System (USCS). The USCS classifies soils on the basis of engineering properties. The system divides soils into three major categories; coarse grained, fine grained and highly organic soils. The soil is then subdivided based on either gradation or plasticity characteristics. The classification excludes particles larger than 75 mm. To aid in quantifying material amounts by weight within the respective grain size fractions the following terms have been included to expand the USCS:

Soil Classification	
Clay	<0.002 mm
Silt	0.002 to 0.06 mm
Sand	0.075 to 4.75 mm
Gravel	4.751o 75 mm
Cobbles	75 to 200 mm
Boulders	>200 mm

Terminology	Proportion
“trace”, sand, etc.	1%to 10%
"some"	10% to 20%
Sandy, Gravelly, etc.	20% to 35%
“and”	>35%
Ex., SAND, SILT, etc.	>35%

Notes:

1. Soil properties, such as strength, gradation, plasticity, structure, etc., dictate the soils engineering behaviour over the grain size fractions;
2. With the exception of soil samples tested for grain size distribution or plasticity, all soil samples have been classified based on visual and tactile observations and is therefore an approximate description.

The following table outlines the qualitative terms used to describe the relative density condition of cohesionless soil:

Cohesionless Soils

Compactness	SPT “N” Value (blows/30cm)
Very Loose	0 to 4
Loose	5 to 10
Compact	11 to 30
Dense	31 to 50
Very Dense	>50

The following table outlines the qualitative terms used to describe the consistency of cohesive soils related to undrained shear strength and SPT, N-Index:

Cohesive Soils

Consistency	Undrained Shear Strength (kPa)	SPT “N” Value (blows/30 cm)
Very Soft	<12.5	< 2
Soft	12.5 to 25	2 to 4
Firm	25 to 50	5 to 8
Stiff	50 to 100	9 to 15
Very Stiff	100 to 200	16 to 30
Hard	> 200	>30

Note: Utilizing the SPT, “N” value to correlate the consistency and undrained shear strength of cohesive soils is very approximate and needs to be used with caution.

Particle Sizes

Constituent	Description	Size (mm)	Size (in)
BOULDERS	Not Applicable	>300	>12
COBBLES	Not Applicable	75 to 300	3 to 12
GRAVEL	Coarse Fine	19 to 75 4.75 to 19	0.75 to 3 (4) to 0.75
SAND	Coarse Medium Fine	2.00 to 4.75 0.425 to 2.00 0.075 to0.425	(10) to (4) (40) to (10) (200) to (40)
SILT/CLAY	Classified by plasticity	< 0.075	< (200)

ROCK CORING

Rock Quality Designation (RQD) is an indirect measure of the number of fractures within a rock mass, Deere et al. (1967). It is the sum of sound pieces of rock core equal to or greater than 100 mm recovered from the core run, divided by the total length of the core run, expressed as a percentage. If the core section is broken due to mechanical or handling, the pieces are fitted together and if 100 mm or greater included in the total sum.

Intact Rock Strength

Intact Strength (Mpa)	Description
< 1	Extremely low strength
1-5	Very low strength
5-25	Low strength
25-50	Medium strength
50-100	High strength
100-250	Very high strength
>250	Extremely high strength

Rock Mass Quality

RQD Classification	RQD Value (%)
Very Poor Quality	<25
Poor Quality	25 to 50
Fair Qualty	50 to 75
Good Quality	75 to 90
Excellent Quality	90 to 100

Rock Mass Weathering

Term	Description
Unweathered (Fresh)	No visible sign of material weathering to discoloration on major discontinuity surfaces.
Slightly Weathered	Discoloration indicates weathering of rock material and discontinuity of surfaces. All the rock material may be discolored by weathering and may be somewhat weaker than its fresh condition.
Moderatly Weathered	Less than half the rock material is decomposed and/or disintegrates to soil. Fresh or discolored rock is present either as a continuous frame work of as core stones.
Highly Weathered	More than half the rock material is decomposed and/or disintegrated to soil. Fresh or discolored rock is present either as a discontinuous frame work or as core stones.
Completely Weathered	All rock material is decomposed and/or disintegrated to soil. The original mass structure is largely intact.
Residual Soil	All rock material is converted to soil. The mass structure and material fabric are destroyed. There is a large change in volume, but the soil has not been significantly transported.

Joint and Foliation Spacing

Description	Spacing
Very Wide	Greater than 3 m
Wide	1 m to 3 m
Moderately Close	0.3 m to 1 m
Close	50 mm to 300 mm
Very Close	Less than 50 mm

Bedding Thickness

Description	Spacing
Very thick	Greater than 2 m
Thick	0.6 m to 2 m
Medium	0.2 m to 0.6 m
Thin	60 mm to 0.2 m
Very thin	20 mm to 60 mm
Laminated	6 to 20 mm
Thinly Laminated	Less than 6 mm

SYMBOLS

General

w_N Natural water content within the soil sample
γ Unit weight
γ' Effective unit weight
γ_D Dry unit weight
γ_{SAT} Saturated unit weight
ρ Density
ρ_s Density of solid particles
ρ_w Density of water
ρ_D Dry density
ρ_{SAT} Saturated density
e Void ratio
n Porosity
S Degree of saturation
E_{50} Fifty percent secant modulus

Consistency

w_L Liquid Limit
w_P Plastric Limit
I_P Plasticity Index
w_S Shrinkage limit
I_L Liquidity index
I_C Consistency index
e_{max} Void ratio in loosest state
e_{min} Void ratio in densest state
I_D Density index (formerly relative density)

Shear Strength

S_u Undrained shear strength parameter (total stress)
c' Effective cohesion intercept
ϕ' Effective friction angle
τ_R Peak shear strength
τ_R Residual shear strength
δ Angle of interface friction
μ Coefficient of friction = tan ϕ'

Consolidation

C_c Compression index (normally consolidated range)
C_r Recompression index (over consolidated range)
m_v Coefficient of volume change
c_v Coefficient of consolidation
T_v Time factor (vertical direction)
U Degree of consolidation
σ_v' Effictive overburden pressure
OCR Overconsolidation ratio

Site Photo Log

Photo 1: General investigation area, note low powerlines on left side of photo which prevented drilling closer to the North Slope. Powerhouse on left.

Photo 2: Retaining wall near road surface, gabion basket wall at slope toe. Powerhouse on right. Image looking from downstream of powerhouse to upstream.

CLIENT Township of Muskoka Lakes	PROJECT Burgess Dam – North Slope Investigation

CONSULTANT

TULLOCH

YYYY-MM-DD	2022-03-08
PREPARED	K. Cheung
DESIGNED	K. Cheung
REVIEWED	E.Giles
APPROVED	

TITLE
Geotechnical Investigation – Site Photos

PROJECT NO.	Phase/Task	Rev.	FIGURE
20-1051		**Rev 0**	**C1**

Photo 3: View of retaining wall behind the fence from road surface.

Photo 4: North Slope with powerhouse and tailrace in background on right. Note abrupt slope change where gabion basket wall exists at break in slope.

CLIENT
Township of Muskoka Lakes

PROJECT
Burgess Dam – North Slope Investigation

CONSULTANT

TULLOCH

YYYY-MM-DD	2022-03-08
PREPARED	K. Cheung
DESIGNED	K. Cheung
REVIEWED	E.Giles
APPROVED	

TITLE
Geotechnical Investigation – Site Photos

PROJECT NO.	Phase/Task	Rev.	FIGURE
20-1051		Rev 0	C2

Photo 5: Gabion wall at toe of North Slope. Note rockfill located at toe of gabion wall above tailrace water level.

Borehole Logs

RECORD OF BOREHOLE No 20-01

METRIC

TULLOCH ENGINEERING

JOB NUMBER 20-1051	LOCATION River Street, Bala, Ontario		ORIGINATED BY JM
CLIENT Township of Muskoka Lakes	DATUM Ground Suface	BOREHOLE TYPE HSA/NQ Diamond Rotary	COMPILED BY JM
DRILLER Landcore Drilling	DATE 2020.09.09	NORTHING 4985600 EASTING 609067	CHECKED BY EG

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
0.00	Ground Surface					
0.00 / -0.13	125 mm ASPHALT					Grinding augers from 0.125 m to 1.47 m. Inferred cobbles to boulders.
0.13	FILL - (SW) SAND, fine to coarse grained, gravelly to some fine to coarse gravel, sub-angular, trace non-plastic fines, brown (PAVEMENT STRUCTURE, Base, Subbase); non-cohesive, moist, dense to compact	1	SS	31		12 82 (6)
		2	SS	20		30 58 (12)
-1.47	Note: - Auger refusal encountered at 1.47 m. - Landcore Drilling switched to NW casing and core barrel.					
1.47 / -2.95	BEDROCK - Granitic Gneiss, fine to medium grained, angled foliation, medium to coarse grained feldspar intrusion, natural vertical and angular jointing with muscovite and calcite deposits within discontinuities, angular and horizontal fractures throughout, slightly weathered, strong rock Note: - SILT infiltration in discontinuity near 2.59 m Run 1: RQD: 83/147 = 56% TCR: 138/147 = 94% SCR: 105/147 = 71%	Run 1	NQ			Rock Core Compressive Strength at 2.3 mbgs = 100.3 MPa
2.95 / -4.50	BEDROCK - Granitic gneiss, fine to medium grained, angled foliation, medium to coarse grained feldspar intrusion, angular and horizontal fractures throughout, unweathered, strong rock Run 2: RQD: 145/155 = 94% TCR: 155/155 = 100% SCR: 155/155 = 100%	Run 2	NQ		∇ (4.12 m)	Rock Core Compressive Strength at 3.9 mbgs = 130.3 MPa
4.50	END OF BOREHOLE Note: - Groundwater was measured at 4.12 m upon completion of investigation. It should be noted that groundwater may not be stabilized upon completion of borehole. - A reduced section sub broke during the attempted removal of a 1.54 m long section of streel casing which became ceased within the borehole. Landcore Drilling was unable to remove this ceased section of casing, therefore it was hammered to 0.2 m below top of asphalt surface, backfilled and abandoned in the borehole.					

Column headings: SOIL PROFILE; SAMPLES; STRAT PLOT; DEPTH (M) 1, 2, 3, 4; DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ POCKET PEN + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100; PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT W_L; WATER CONTENT (%) 20 40 60; UNIT WEIGHT γ kN/m³

200 + : Numbers refer to Field Vane Over Limit +3, ×3: Numbers refer to Sensitivity ○3% STRAIN AT FAILURE

TULLOCH ENGINEERING

RECORD OF BOREHOLE No 20-02

METRIC

JOB NUMBER 20-1051 LOCATION River Street, Bala, Ontario ORIGINATED BY JM

CLIENT Township of Muskoka Lakes DATUM Ground Suface BOREHOLE TYPE HSA COMPILED BY JM

DRILLER Landcore Drilling DATE 2020.09.09 NORTHING 4985600 EASTING 609067 CHECKED BY EG

ELEV DEPTH	SOIL PROFILE DESCRIPTION	STRAT PLOT	SAMPLES NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	DEPTH (M)	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (○ POCKET PEN, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE; 20 40 60 80 100)	PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L — WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
0.00	Ground Surface										
0.00 -0.13	125 mm ASPHALT										
0.13	FILL - (SW) SAND, fine to coarse grained, gravelly to some fine to coarse gravel, sub-angular, trace non-plastic fines, brown (PAVEMENT STRUCTURE, Base, Subbase); non-cohesive, moist, dense to compact		1	SS	29						
			2	SS	>50/ 2"		1				
-1.24 1.24	END OF BOREHOLE Note: - Spoon and auger refusal encountered at 1.24 m. Inferred bedrock surface - Groundwater was not encountered upon completion of investigation. It should be noted that groundwater may not be stabilized upon completion of borehole.										

200 + : Numbers refer to Field Vane Over Limit

+ 3, × 3: Numbers refer to Sensitivity

○ 3% STRAIN AT FAILURE

TULLOCH ENGINEERING

RECORD OF BOREHOLE No 20-03

METRIC

JOB NUMBER 20-1051 | LOCATION River Street, Bala, Ontario | ORIGINATED BY JM

CLIENT Township of Muskoka Lakes | DATUM Ground Suface | BOREHOLE TYPE HSA | COMPILED BY JM

DRILLER Landcore Drilling | DATE 2020.09.09 | NORTHING 4985600 | EASTING 609067 | CHECKED BY EG

ELEV DEPTH	SOIL PROFILE – DESCRIPTION	STRAT PLOT	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	DEPTH (M)	REMARKS & GRAIN SIZE DISTRIBUTION (%)
0.00	Ground Surface							
0.00 -0.13	125 mm ASPHALT							
0.13	FILL - (SW) SAND, fine to coarse grained, gravelly to some fine to coarse gravel, sub-angular, trace non-plastic fines, brown (PAVEMENT STRUCTURE, Base, Subbase); non-cohesive, moist, dense to compact		1	SS	33			Grinding experienced throughout auger advancement from 0.125 m to 1.78 m. Inferred cobbles to boulders.
			2	SS	30		1	
-1.78			3	SS	>50/ 2"			
1.78	END OF BOREHOLE Note: - Spoon and auger refusal encountered at 1.78 m. Inferred bedrock surface - Groundwater was not encountered upon completion of investigation. It should be noted that groundwater may not be stabilized upon completion of borehole.							

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: ○ POCKET PEN, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE — 20 40 60 80 100

PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L

WATER CONTENT (%): 20 40 60

UNIT WEIGHT γ kN/m^3

GR SA SI CL

200 + : Numbers refer to Field Vane Over Limit

$+^3, \times^3$: Numbers refer to Sensitivity

○ 3% STRAIN AT FAILURE

Rock Core Photos

Retrieved Rock Core at Borehole Location

BH-20-01: Run 1 and Run 2 – 1.47 m to 4.50 m

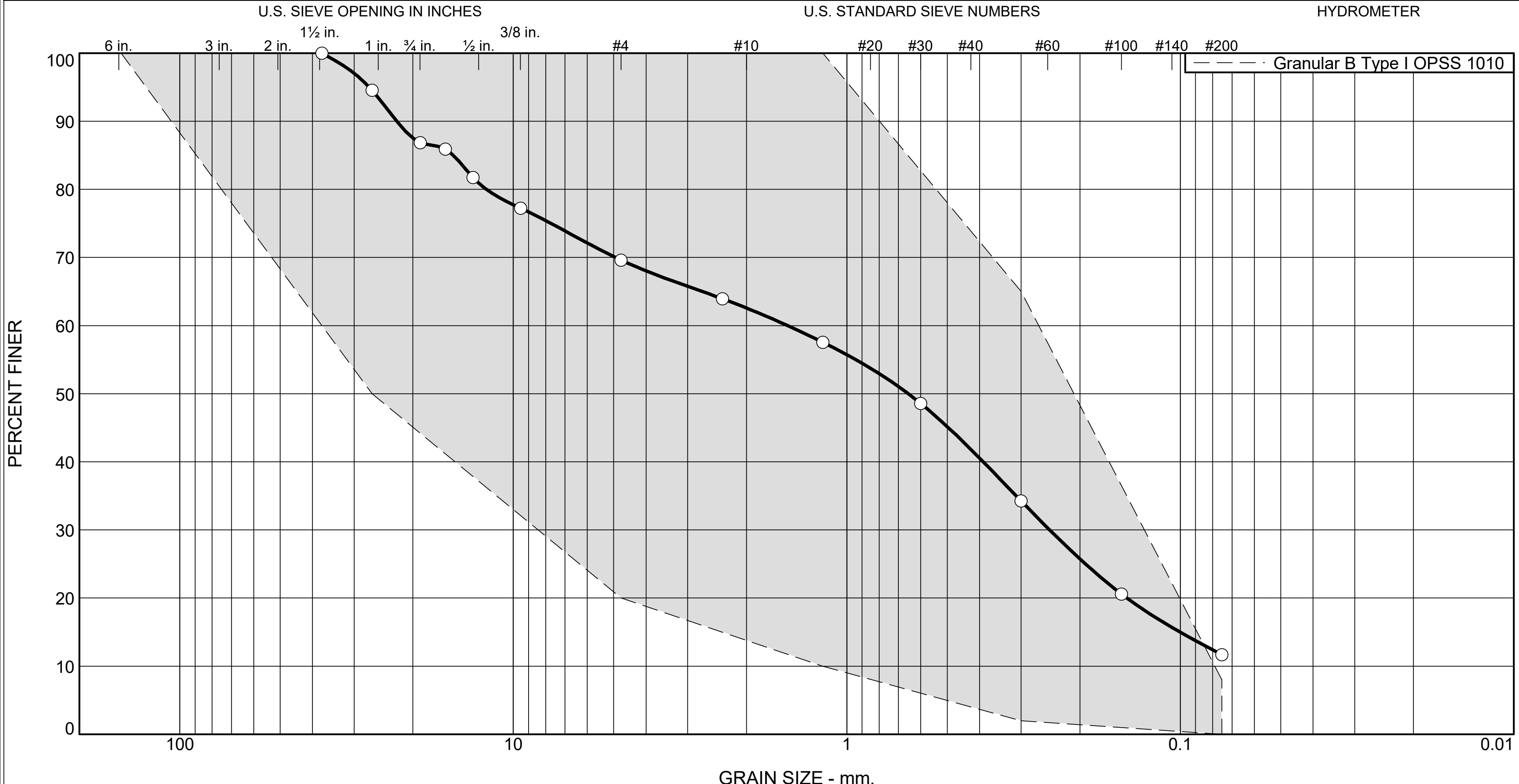
Top of Bedrock

Bottom of Core

CLIENT		PROJECT			
Township of Muskoka Lakes		Burgess Dam – North Slope Investigation			
CONSULTANT		TITLE			
TULLOCH	YYYY-MM-DD: 2022-03-08	Rock Core Photos – BH-20-01			
	PREPARED: KC				
	DESIGN: KC				
	REVIEW: EG	PROJECT No.	Phase / Task	Rev.	Figure
	APPROVED: EG	20-1051		0	E-1

Laboratory Data

Particle Size Distribution Report



	% +3"	% Gravel		% Sand			% Fines
		Coarse	Fine	Coarse	Medium	Fine	Silt
○	0.0	13.1	17.3	7.0	20.8	30.1	11.7

Identification	Date Sampled	Date Received	Date Tested
Source of Sample: BH-21-01 Depth: 0.9m - 1.5m Sample Number: SS2	Sept 9, 2020		Feb 25, 2022

Client Township of Muskoka Lakes

Project Burgess Dam

Project No. 20-1051 Figure

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Daren.Stadnisky@TULLOCH.ca

Tested By: T. Linley

GRAIN SIZE DISTRIBUTION TEST DATA

2022-03-01

Client: Township of Muskoka Lakes
Project: Burgess Dam
Project Number: 20-1051
Location: BH-21-01
Depth: 0.9m - 1.5m **Sample Number:** SS2
Date Sampled: Sept 9, 2020 **Date Tested:** Feb 25, 2022
Tested by: T. Linley
Material specification: Granular B Type I OPSS 1010

Sieve Test Data

Post #200 Wash Test Weights (grams): **Dry Sample and Tare =** 778.00
Tare Wt. = 163.30
Minus #200 from wash = 8.2%

Dry Sample and Tare (grams)	Tare (grams)	Sieve Opening Size	Weight Retained (grams)	Sieve Weight (grams)	Percent Finer	Lower Spec. Limit, %	Upper Spec. Limit, %	Deviation From Spec., %
832.80	163.30	37.5mm	0.00	0.00	100.0			
		26.5mm	36.60	0.00	94.5	50.0	100.0	
		19mm	51.40	0.00	86.9			
		16mm	6.30	0.00	85.9			
		13.2mm	28.00	0.00	81.7			
		9.5mm	30.10	0.00	77.2			
		#4	51.10	0.00	69.6	20.0	100.0	
		#8	38.00	0.00	63.9			
		#16	42.80	0.00	57.5	10.0	100.0	
		#30	60.10	0.00	48.6			
		#50	95.80	0.00	34.2	2.0	65.0	
		#100	91.50	0.00	20.6			
		#200	59.60	0.00	11.7	0.0	8.0	*+3.7*

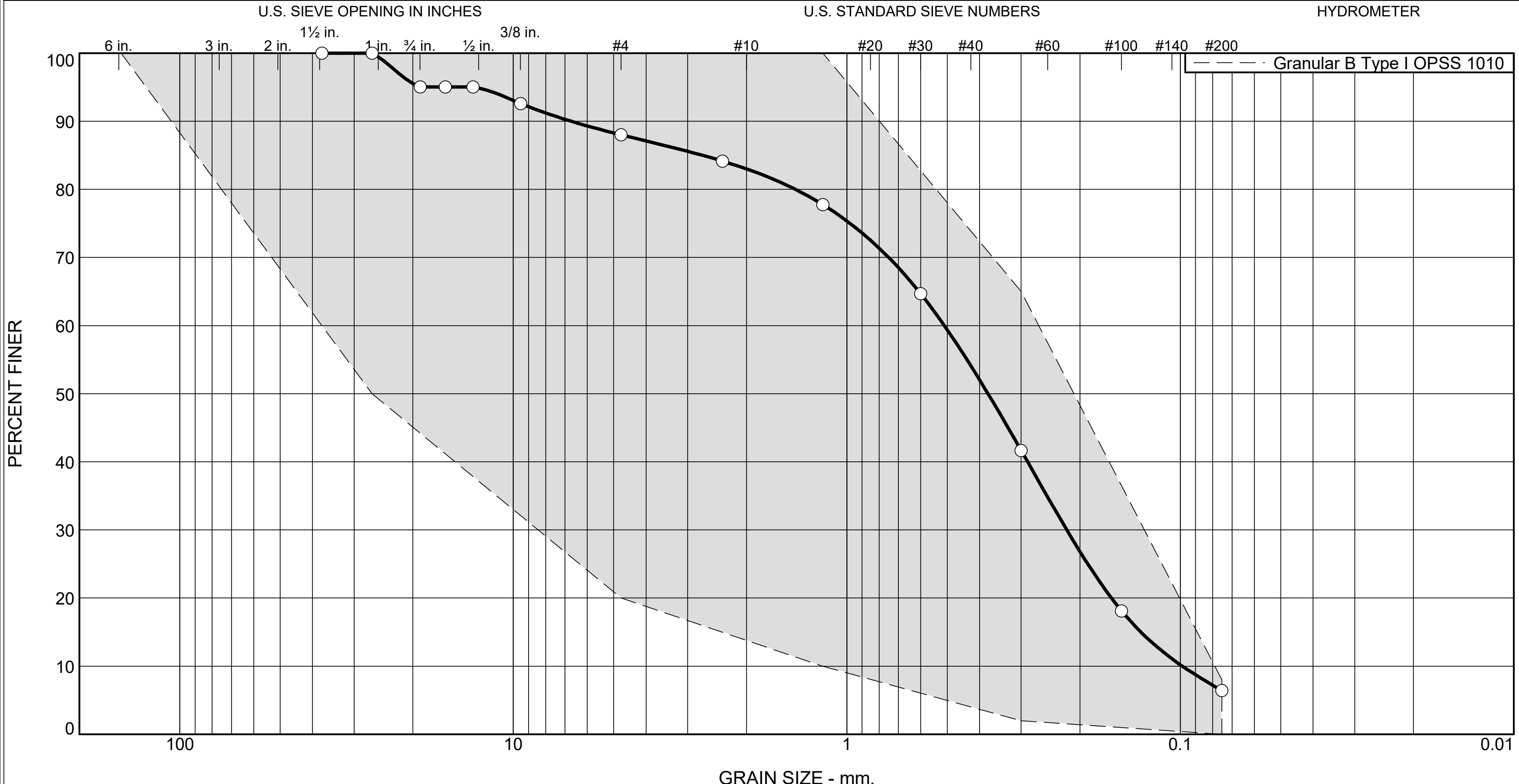
Fractional Components

Cobbles	Gravel			Sand				Fines		
	Coarse	Fine	Total	Coarse	Medium	Fine	Total	Silt	Clay	Total
0.0	13.1	17.3	30.4	7.0	20.8	30.1	57.9			11.7

D_5	D_{10}	D_{15}	D_{20}	D_{30}	D_{40}	D_{50}	D_{60}	D_{80}	D_{85}	D_{90}	D_{95}
		0.1001	0.1446	0.2464	0.3900	0.6544	1.5061	12.0512	15.1429	22.3752	27.0349

Fineness Modulus
3.41

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines
	Coarse	Fine	Coarse	Medium	Fine	Silt
0.0	4.9	7.1	5.0	28.9	47.7	6.4

Identification	Date Sampled	Date Received	Date Tested
Source of Sample: BH-21-02 Depth: 0.2m - 0.8m Sample Number: SS1	Sept 9, 2020		Feb 25, 2022

Client Township of Muskoka Lakes

Project Burgess Dam

Project No. 20-1051 Figure

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F. 705 949.9606
TF. 866 806.6602
Daren.Stadnisky@TULLOCH.ca

Tested By: T. Linley

GRAIN SIZE DISTRIBUTION TEST DATA

2022-03-01

Client: Township of Muskoka Lakes
Project: Burgess Dam
Project Number: 20-1051
Location: BH-21-02
Depth: 0.2m - 0.8m **Sample Number:** SS1
Date Sampled: Sept 9, 2020 **Date Tested:** Feb 25, 2022
Tested by: T. Linley
Material specification: Granular B Type I OPSS 1010

Sieve Test Data

Post #200 Wash Test Weights (grams): **Dry Sample and Tare =** 879.00
Tare Wt. = 151.50
Minus #200 from wash = 4.1%

Dry Sample and Tare (grams)	Tare (grams)	Sieve Opening Size	Weight Retained (grams)	Sieve Weight (grams)	Percent Finer	Lower Spec. Limit, %	Upper Spec. Limit, %	Deviation From Spec., %
910.20	151.50	37.5mm	0.00	0.00	100.0			
		26.5mm	0.00	0.00	100.0	50.0	100.0	
		19mm	37.60	0.00	95.0			
		16mm	0.00	0.00	95.0			
		13.2mm	0.00	0.00	95.0			
		9.5mm	18.50	0.00	92.6			
		#4	34.90	0.00	88.0	20.0	100.0	
		#8	29.30	0.00	84.1			
		#16	48.50	0.00	77.8	10.0	100.0	
		#30	99.30	0.00	64.7			
		#50	174.50	0.00	41.7	2.0	65.0	
		#100	178.70	0.00	18.1			
		#200	88.80	0.00	6.4	0.0	8.0	

Fractional Components

Cobbles	Gravel			Sand				Fines		
	Coarse	Fine	Total	Coarse	Medium	Fine	Total	Silt	Clay	Total
0.0	4.9	7.1	12.0	5.0	28.9	47.7	81.6			6.4

D_5	D_{10}	D_{15}	D_{20}	D_{30}	D_{40}	D_{50}	D_{60}	D_{80}	D_{85}	D_{90}	D_{95}
	0.0986	0.1313	0.1609	0.2189	0.2869	0.3771	0.5100	1.4264	2.7128	6.7118	13.0360

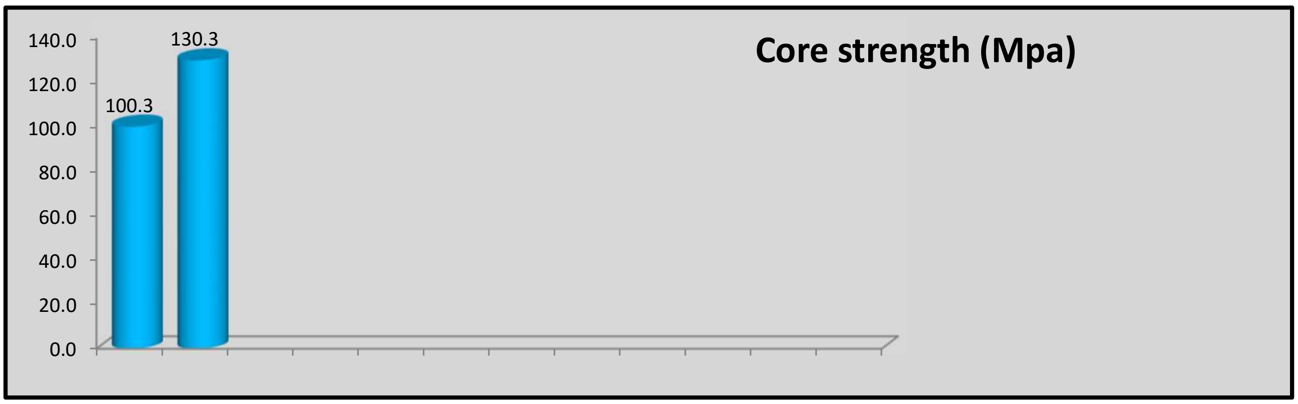
Fineness Modulus	C_u	C_c
2.38	5.17	0.95

CSA A283 Certified Laboratory for Concrete Testing
CCIL Certified Laboratory for Aggregates and Asphalt Testing
CSA/CCIL Certified Technicians

Rock Core Compressive Strength Report

PROJECT: Burgess Dam
DATE SAMPLED: September 9, 2020
DATE TESTED: February 25, 2022
CONTRACT: 20-1051
RUN BY: J. Draper
SOURCE: Boreholes

Sample Location	Run #	Distance from top of run (cm)	Height (mm)	Diameter (mm)	L/D Ratio	Correction Factor	Peak Load (lbs)	Compressive Stength (Mpa)
BH-01	1	81	94.62	47.35	2.0	1.0	39700	100.3
BH-01	2	97	94.68	47.41	2.0	1.0	51700	130.3

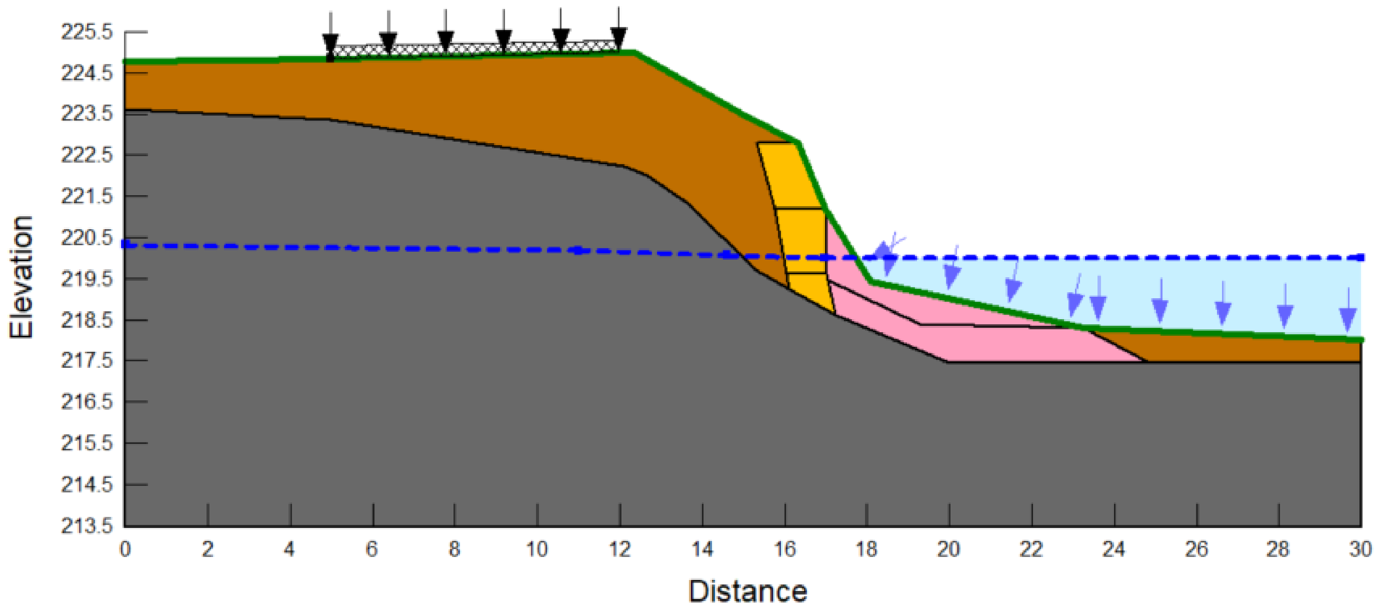


REMARKS:

CLIENT: Township of Muskoka Lakes

Slope Stability Results

Color	Name	Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Phi-B (°)	Piezometric Line
	Bedrock	Bedrock (Impenetrable)					1
	Gabion Baskets	Mohr-Coulomb	20	30	38	0	1
	Rockfill	Mohr-Coulomb	20	0	38	0	1
	Sandy Soil	Mohr-Coulomb	19	0	35	0	1



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CLIENT
Township of Muskoka Lakes

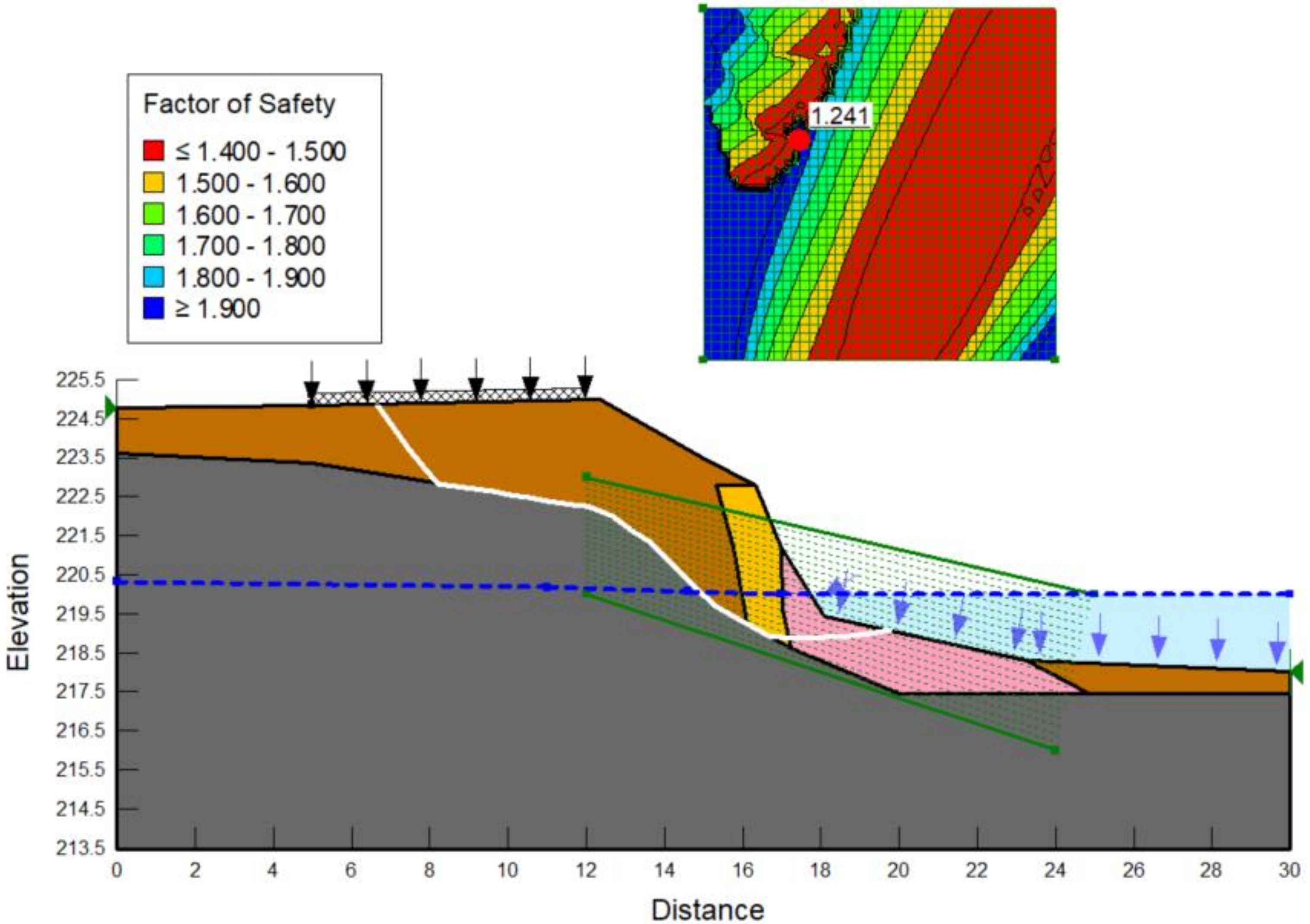
PROJECT
Burgess Dam – North Slope Investigation

CONSULTANT
TULLOCH

YYYY-MM-DD	2022-03-08
PREPARED	KC
DESIGN	KC
REVIEW	EG
APPROVED	GL

TITLE
North Slope
Geostudio LE Model Geometry and Parameters

PROJECT No.	Phase / Task	Rev.	Figure
20-1051	-	A	G-1



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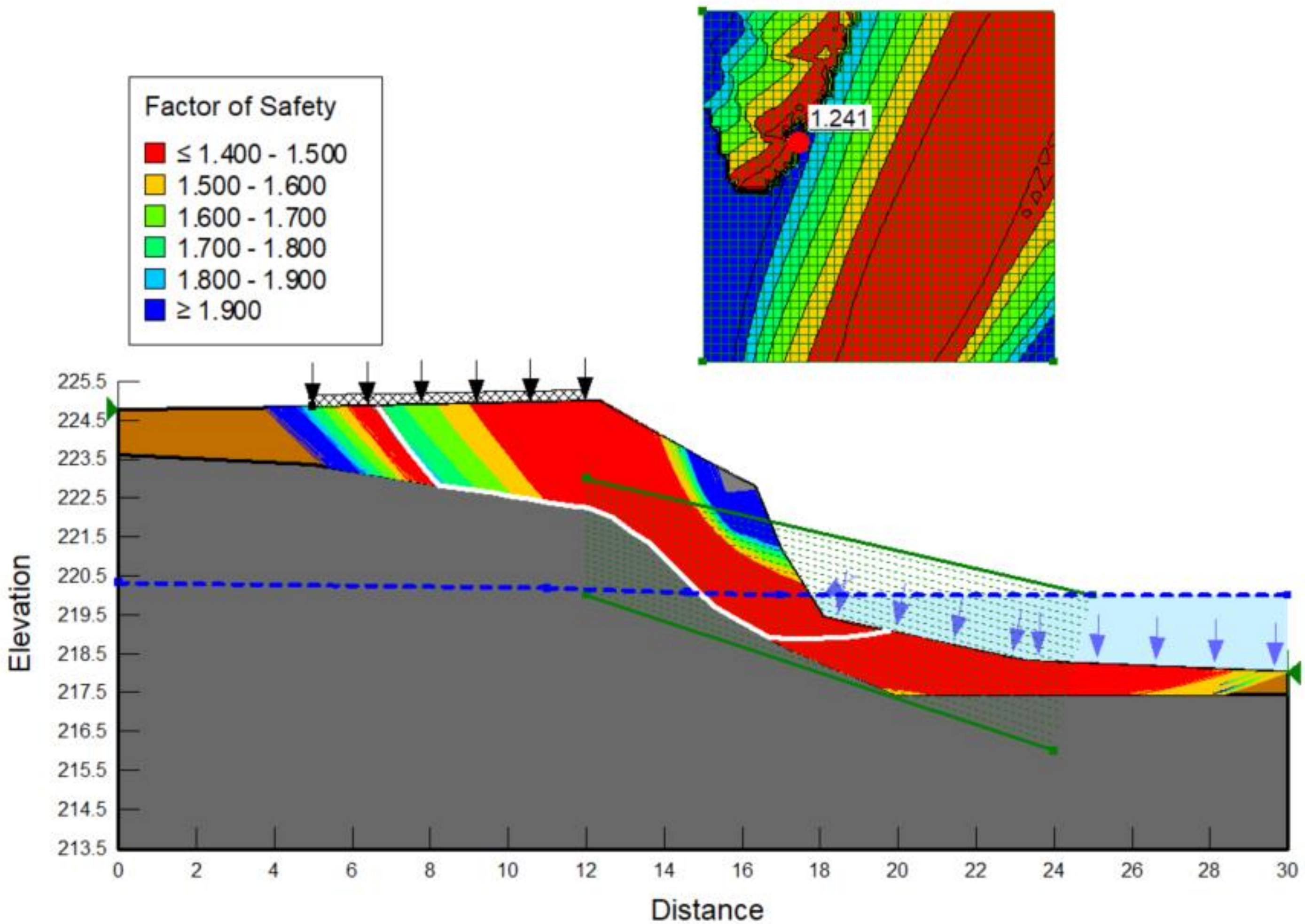
CLIENT	PROJECT
Township of Muskoka Lakes	Burgess Dam – North Slope Investigation

CONSULTANT: TULLOCH

YYYY-MM-DD	2022-03-08
PREPARED	KC
DESIGN	KC
REVIEW	EG
APPROVED	GL

TITLE
North Slope
Geostudio LE Model Results

PROJECT No.	Phase / Task	Rev.	Figure
20-1051	-	A	G-2



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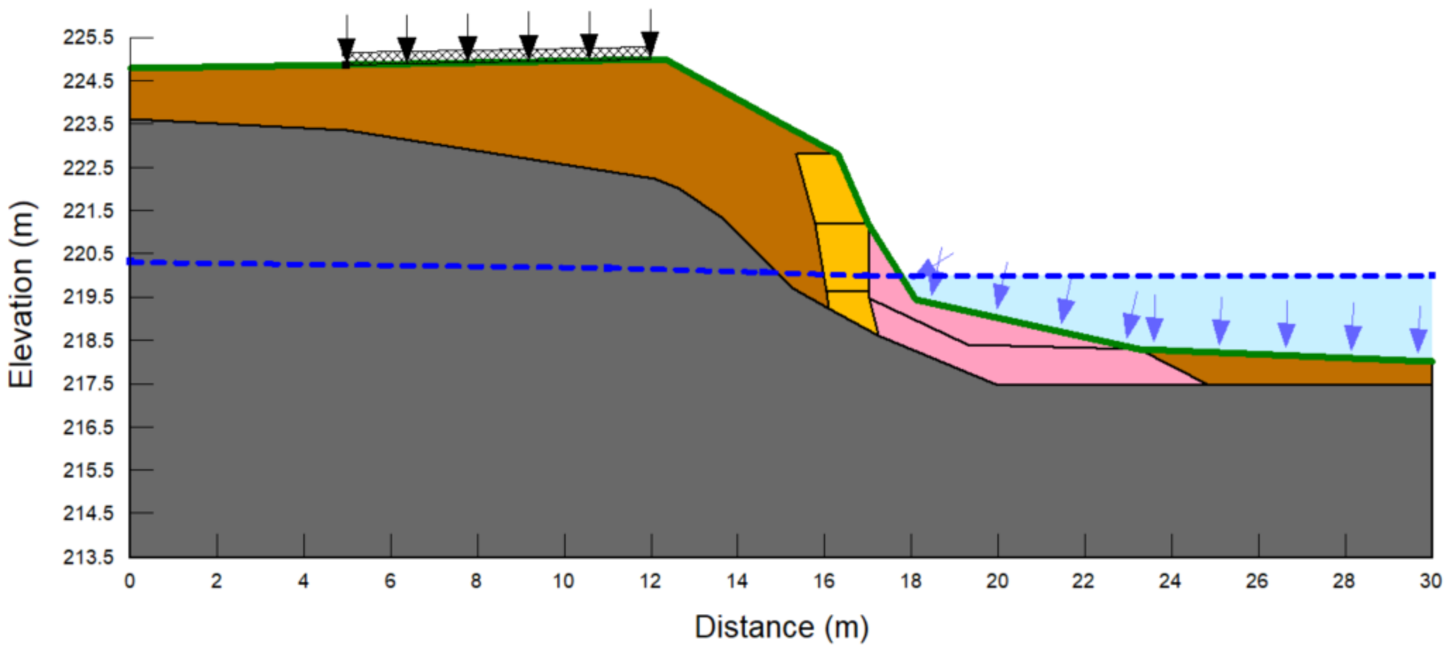
CLIENT	Township of Muskoka Lakes	PROJECT	Burgess Dam – North Slope Investigation
CONSULTANT	TULLOCH	TITLE	North Slope Geostudio LE Model Results
YYYY-MM-DD	2022-03-08	PROJECT No.	20-1051
PREPARED	KC	Phase / Task	-
DESIGN	KC	Rev.	A
REVIEW	EG	Figure	G-3
APPROVED	GL		

Color	Name	Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Phi-B (°)	Piezometric Line
	Bedrock	Bedrock (Impenetrable)					1
	Gabion Baskets	Mohr-Coulomb	20	0	38	0	1
	Rockfill	Mohr-Coulomb	20	0	38	0	1
	Sandy Soil	Mohr-Coulomb	19	0	35	0	1



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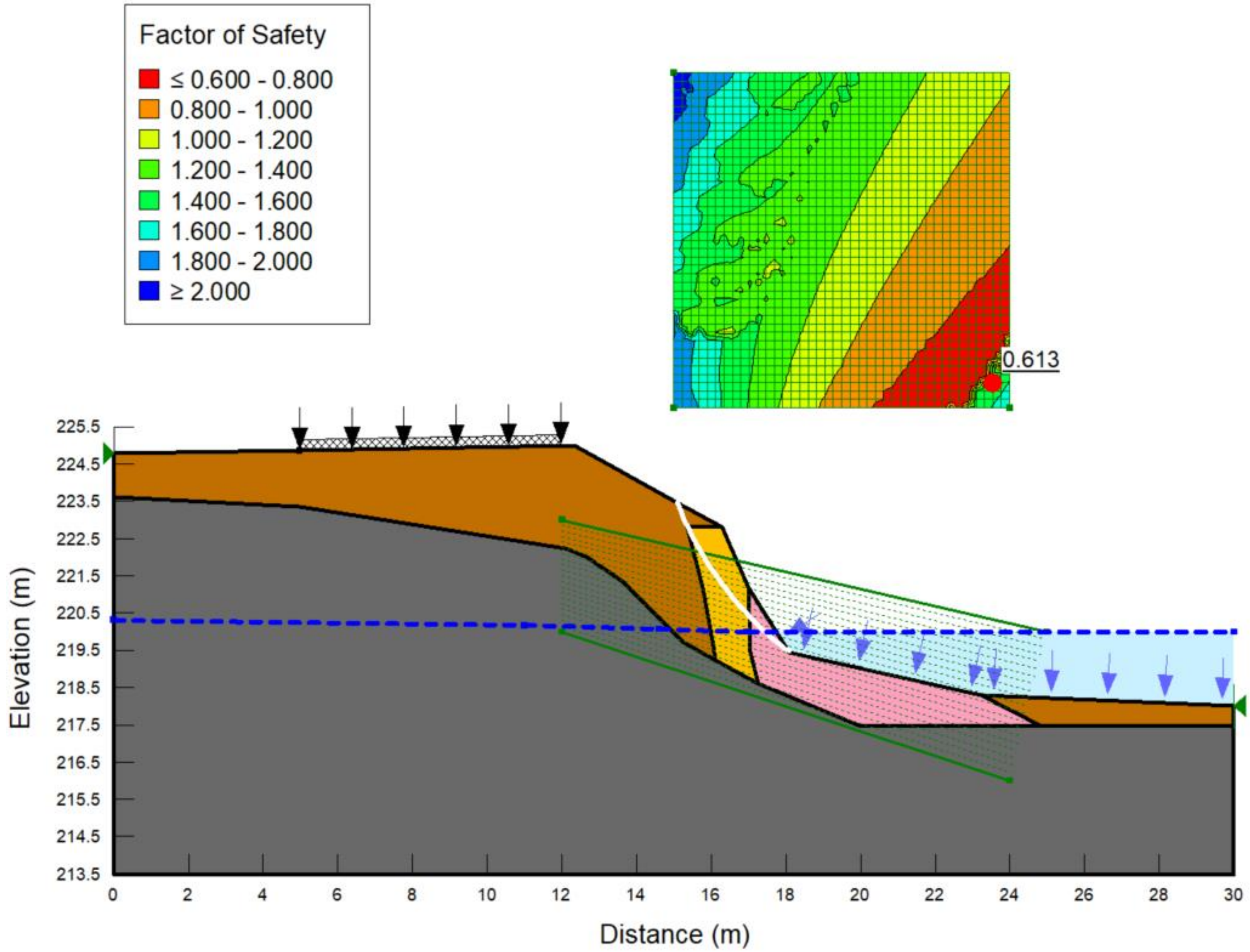
CLIENT	Township of Muskoka Lakes	PROJECT	Burgess Dam – North Slope Investigation
CONSULTANT	TULLOCH	TITLE	North Slope – Failed Gabion Meshing Geostudio LE Model Geometry and Parameters
YYYY-MM-DD	2022-03-08	PROJECT No.	20-1051
PREPARED	KC	Phase / Task	-
DESIGN	KC	Rev.	A
REVIEW	EG	Figure	G-4
APPROVED	GL		



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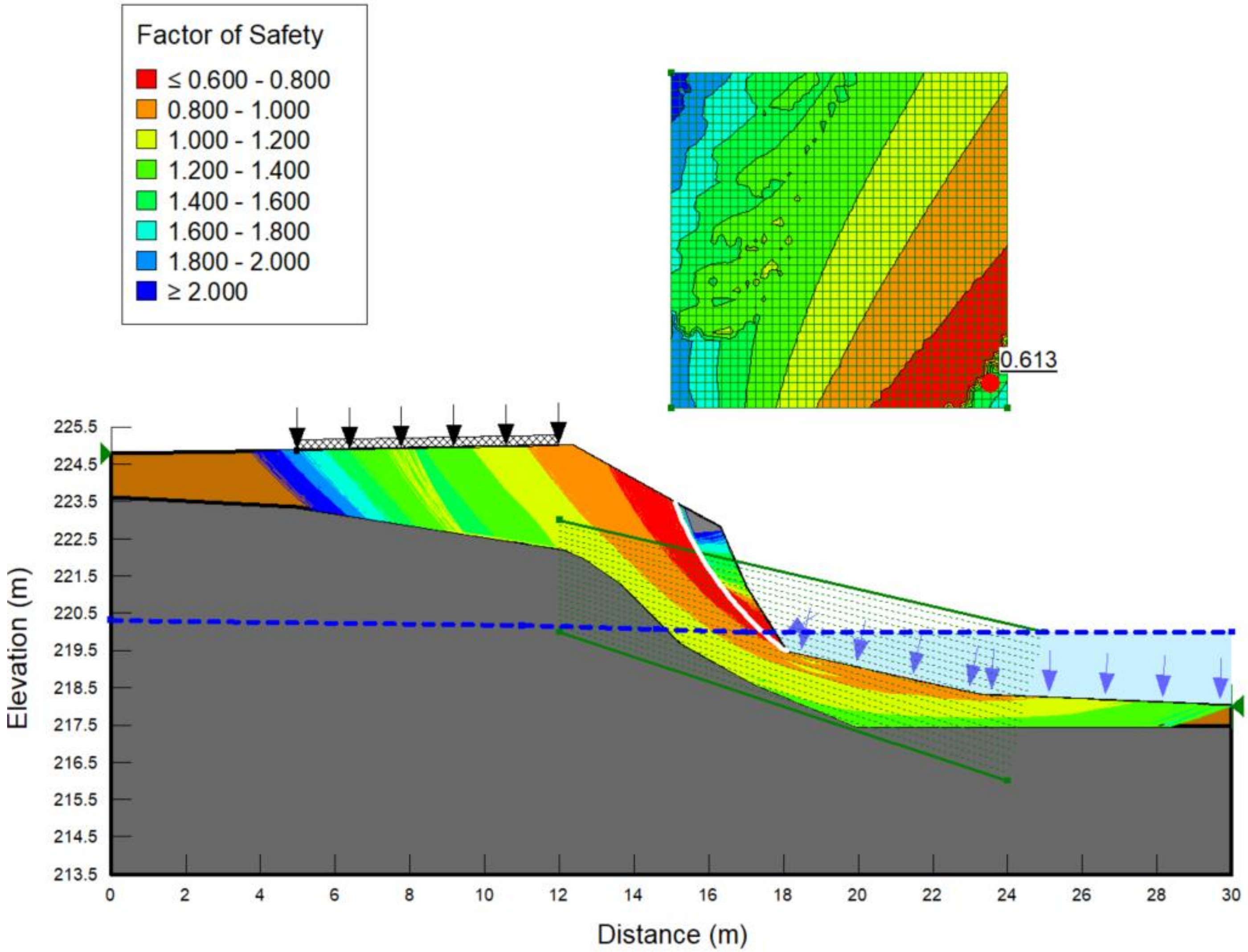
CLIENT Township of Muskoka Lakes	PROJECT Burgess Dam – North Slope Investigation

CONSULTANT		
TULLOCH	YYYY-MM-DD	2022-03-08
	PREPARED	KC
	DESIGN	KC
	REVIEW	EG
	APPROVED	GL

TITLE
North Slope – Failed Gabion Meshing
Geostudio LE Model Results

PROJECT No.	Phase / Task	Rev.	Figure
20-1051	-	A	G-5



Directory: \\stoneycreek2\Projects\Projects\2020\20-1051 - Burgess Dam\01 Engineering\North Slope Investigation\Calculations\

CLIENT	PROJECT
Township of Muskoka Lakes	Burgess Dam – North Slope Investigation

CONSULTANT

TULLOCH

YYYY-MM-DD	2022-03-08
PREPARED	KC
DESIGN	KC
REVIEW	EG
APPROVED	GL

TITLE

North Slope – Failed Gabion Meshing
Geostudio LE Model Results

PROJECT No.	Phase / Task	Rev.	Figure
20-1051	-	A	G-6

Givens and Assumptions

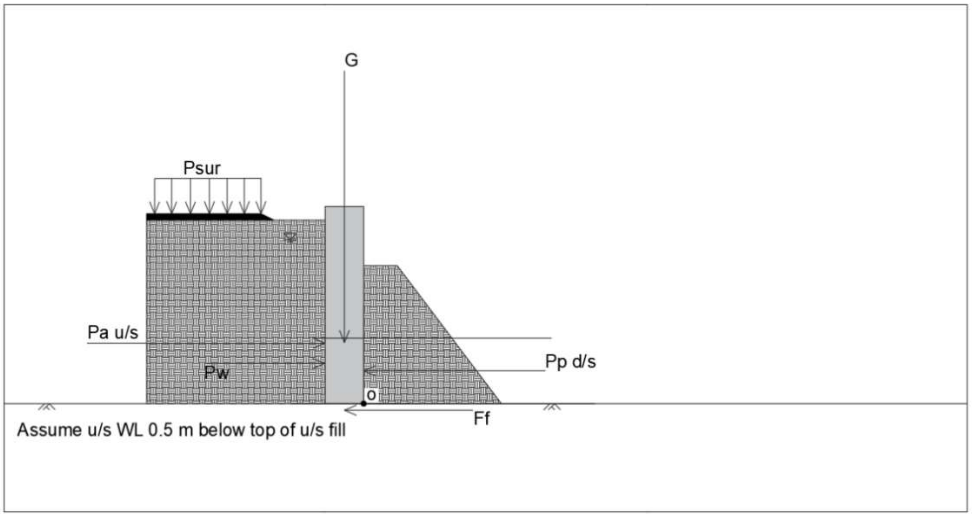
Geometry Input Parameters

Max. Wall Height	H	3.66	m
Dam Base width	t	0.30	m
Height of the u/s fill	hfus	3.35	m
Height of the d/s fill	hfds	2.44	m
Height of u/s water	hw	3.35	m
Traffic Surcharge Loading	Psur	20	kPa

Soil/Rock Input Parameters

Unit weight-Unreinforced Concrete	γc	23.58	kN/m^3
Unit weight-u/s and d/s Fill	γf	19	kN/m^3
Unit weight of water	γw	9.8	kN/m^3
Friction angle- u/s and d/s fill	ϕ'f	35	degree
Friction angle- Concrete-to-rock interface	ϕ'c-R	38	degree
Active Earth Pressure Coeff.	ka	0.27	-
Passive Earth Pressure Coeff.	kp	3.69	-

WL 0.5m below Top of U/S Fill - U/S to D/S Slide Direction



*N.T.S

Calculation

Force (kN)	FBD ID	Force (kN)	Moment Arm to "O" (m)	Moment (kN.m)
Traffic Surcharge Load	Pt	5.42	1.68	9.09
u/s Water Pressure	Pw	39.88	0.95	37.92
u/s Active Earth Pressure	Pa u/s	28.94	1.12	32.34
d/s Passive Earth Pressure	Pp d/s	208.44	0.81	-169.42
Gravity Force of Concrete dam	G	26.29	0.15	-4.01
Uplift Force	n/a	0.00	0.00	0.00
Friction Force-Concrete-to-Rock	Ff	14.27	0.00	0.00

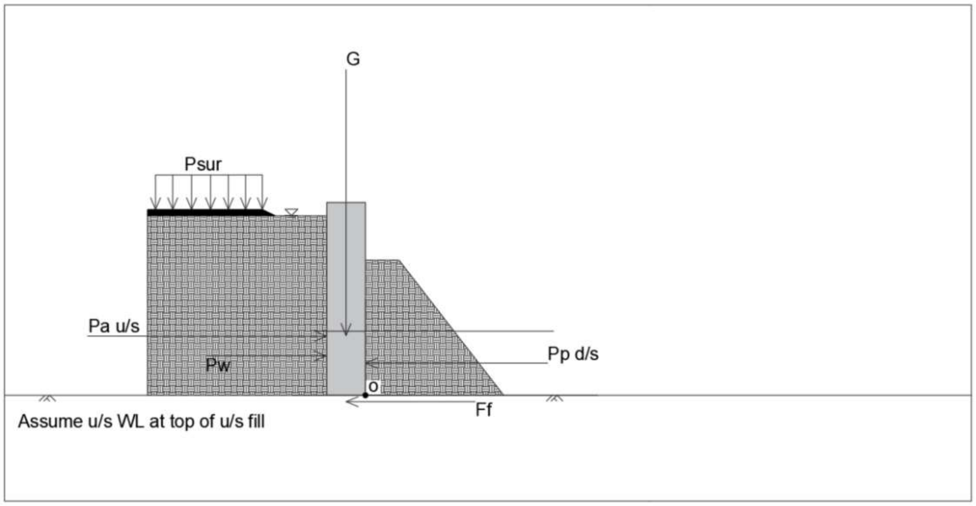
Result

	∑Applied Force (kN)	∑Resistive Force (kN)	FOS		Required FOS
Sliding	74.2	222.7	3.0	OK	1.5

	∑OT Moment (kN*m)	∑ Anti–OT Moment (kN*m)	FOS		Required FOS
Overturning	79.4	-173.4	2.2	OK	2.0

Calculated By: KC
Checked By: EG

WL at Top of U/S Fill - U/S to D/S Slide Direction



*N.T.S

Calculation

Force (kN)	FBD ID	Force (kN)	Moment Arm to "O" (m)	Moment (kN.m)
Traffic Surcharge Load	Pt	5.42	1.68	9.09
u/s Water Pressure	Pw	39.88	0.95	37.92
u/s Active Earth Pressure	Pa u/s	28.94	1.12	32.34
d/s Passive Earth Pressure	Pp d/s	208.44	0.81	-169.42
Gravity Force of Concrete dam	G	26.29	0.15	-4.01
Uplift Force	n/a	0.00	0.00	0.00
Friction Force-Concrete-to-Rock	Ff	14.27	0.00	0.00

Result

Sliding	∑Applied Force (kN)	∑Resistive Force (kN)	FOS	OK	Required FOS
	89.4	222.7	2.5		1.5

Overturning	∑OT Moment (kN*m)	∑ Anti–OT Moment (kN*m)	FOS	Not OK	Required FOS
	103.0	-173.4	1.7		2.0

Calculated By: KC
Checked By: EG

Notice to Reader

NOTICE TO READER

This factual Report has been prepared by TULLOCH Engineering Inc. (‘TULLOCH’) for the sole and exclusive use of the Township of Muskoka Lakes. (the ‘Client’) to support the rehabilitation of the north slope located downstream of the Burgess 1 Dam facility along River Street (the ‘Development’) in Bala, Ontario (the ‘Site’). The Report shall not be used for any other purpose, or provided to, relied upon or used by any third party without the express written consent of TULLOCH.

A limited number of boreholes were advanced at the Site; and as such, the information collected and presented herein applies to the borehole locations only. The subsurface conditions between boreholes can change and accordingly any use of the data contained in this Report should take into consideration the nature of the materials and potential variation between boreholes.

This Report contains opinions, conclusions and recommendations made by TULLOCH using professional judgment and reasonable care for the purpose preliminary assessment for the Development. Use of or reliance on this report by the Client is subject to the following conditions:

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b) the report being read in its entirety. TULLOCH is not responsible for the use of portions of the report without reference to the entire report;
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