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Slope Stability Assessment
Ottawa-Carleton Detention Centre
Parking Lot Expansion - 2244 Innes Road

Ottawa, Ontario

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Submitted to:

Jp2g Consultants Inc.
1150 Morrison Drive, Suite 410
Ottawa, Ontario
K2H 8S9

Slope Stability Assessment
Ottawa-Carleton Detention Centre
Parking Lot Expansion - 2244 Innes Road
Ottawa, Ontario

July 29, 2022
Project: 100009.037 R0

GEMTEC Consulting Engineers and Scientists Limited
32 Steacie Drive
Ottawa, ON, Canada
K2K 2A9

July 29, 2022 File: 100009.037 R0

Jp2g Consultants Inc.
1150 Morrison Drive, Suite 410
Ottawa, Ontario
K2H 8S9

Attention: Mr. David Nguyen, P.Eng., ing.

Re: Slope Stability Assessment
Ottawa-Carleton Detention Centre
Parking Lot Expansion - 2244 Innes Road
Ottawa, Ontario

Please find enclosed our final geotechnical site investigation and slope stability assessment report for the Ottawa-Carleton Detention Centre parking lot expansion project in Ottawa, Ontario. This report has been prepared in general accordance with GEMTEC's proposal dated February 9, 2022.

Luc Bouchard, P.Eng., ing.
Geotechnical Engineer

Graeme Skinner, PhD., P.Eng.
Senior Geotechnical Engineer,

LB/GDS

Enclosures
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TABLE OF CONTENTS

LIST OF TABLES

LIST OF APPENDICES

1.0 INTRODUCTION

This report presents the results of the geotechnical site investigation and slope stability assessment completed by GEMTEC Consulting Engineers and Scientists Limited (GEMTEC) in support of the parking lot expansion project at the Ottawa-Carleton Detention Centre (OCDC) located at 2244 Innes Road in Ottawa, Ontario (referred to herein as the ‘subject site’).

It is our understanding that the proposed project consists of the rehabilitation of the storm water management system, as well as the construction of a new pavement structure for the parking area (the parking area is currently gravel surfaced) at the site. It is further understood that the proposed finished grades associated with the project will not be increased significantly, i.e., within +/- 0.1 m, from the current levels / elevations.

It is also understood that there are concerns with regards to the proposed works impacting the existing stability of the slope located on the south side of the property.

2.0 BACKGROUND

2.1 Description of Study Area

The site is located within an existing developed area above a ravine / creek located to the south. The site itself is generally flat and consists of existing structures, paved, gravel and grassed / landscaped surfaces. The area to the south, beyond the limits of the site, slopes into an existing vegetated ravine / creek area. The slopes are vegetated with grass and young to mature trees, and active surficial erosion and some mirror erosion along the banks of the watercourse were observed.

Based on available topographic information, the southern most limit of the proposed development area is located approximately 60 m from the crest of the existing slope at its closest point.

2.2 Review of Geological Information

Available subsurface information of the area indicates that the site is underlain by approximately 25 m to 50 m of clay overlying bedrock. As such, the slopes to the south of the site are likely formed in local sensitive clay soils, which can be of risk for instability. It should be noted that based on the Ontario Water Well Records for a well located in the vicinity of the site, that the bedrock is anticipated to be located about 33 m below ground surface.

3.0 METHODOLOGY

A site reconnaissance was carried out by a member of our geotechnical engineering staff on April 20, 2022. At that time, the access to the site was assessed and the overall slope conditions were visually observed. The geometry of the slope on the south side of the property closest to the proposed works were measured at critical locations using GPS and manual surveying equipment.

In order to assess the subsurface conditions at the site, one (1) borehole and one (1) seismic cone penetration test (SCPT) hole were advanced on June 15 and 16, 2022 near the crest of the slope in the area of interest (see Borehole and Test Hole Location Plan, Figure A1). The borehole and test hole were advanced using a track mounted drill rig supplied and operated by CCC Geotechnical and Environmental Drilling Ltd. The borehole was advanced to a depth of approximately 25 m below ground surface, was terminated in the local clay deposit, and did not encounter bedrock refusal. The SCPT hole was advanced to a depth of about 26.2 m and was terminated due to the cone exceeding the inclination tolerance.

The fieldwork was observed by members of our geotechnical engineering staff who directed the field operations, observed the in-situ testing and logged the samples, borehole and SCPT. Standard penetration tests were carried out during borehole drilling within the overburden deposits and samples of the soils encountered were recovered using drive open sampling equipment. In-situ vane shear testing was carried out in the borehole to measure the undrained shear strength with depth of the clay deposits. A well screen (i.e., piezometer) was installed in the borehole to assist in measuring the groundwater level.

Following the borehole drilling work, the soil samples were returned to our laboratory for examination by a geotechnical engineer. Selected samples of the soil were submitted for classification testing for water content, grain size distribution and Atterberg limits.

The Record of Borehole sheet and the SCPT Output log are provided in Appendices B and C, respectively. The results of the soil laboratory classification testing are provided in Appendix D. The approximate locations of the boreholes are shown on the Borehole and Test Hole Location Plan, Figure A1 in Appendix A.

The borehole and test hole locations were selected by GEMTEC personnel and positioned at the site relative to existing site features. The ground surface elevations at the borehole and test hole were determined using a Trimble R10 GPS survey instrument. The elevations are referenced to geodetic datum.

4.0 SUBSURFACE CONDITIONS AND SLOPE GEOMETRY

4.1 General

As previously indicated, the soil and groundwater conditions identified in the borehole and SCPT hole are provided on the appended Record of Borehole sheets and SCPT output log (Appendices B and C, respectively). The Records of Borehole and Test Hole indicate the subsurface conditions at the specific test locations only. Boundaries between zones on the records are often not distinct, but rather are transitional and have been interpreted. The precision with which subsurface conditions are indicated depends on the method of drilling / testing, the frequency and recovery of samples, the method of sampling, and the uniformity of the subsurface conditions. Subsurface conditions at other than the test locations may vary from the conditions encountered in the borehole or test hole. In addition to soil variability, fill of variable physical and chemical composition can be present over portions of the site or on adjacent properties.

The groundwater conditions described in this report refer only to those observed at the place and time of observation noted in the report. These conditions may vary due to on-going groundwater recovery, typically seasonally, or as a consequence of construction activities in the area.

The soil descriptions in this report are based on commonly accepted methods of classification and identification employed in geotechnical practice. Classification and identification of soil involves judgement and GEMTEC does not guarantee descriptions as exact but infers accuracy to the extent that is common in current geotechnical practice.

The following presents an overview of the subsurface conditions encountered in the borehole and test hole advanced during this investigation.

4.2 Topsoil

A surficial layer of topsoil was encountered at the borehole location. The topsoil has a thickness of about 50 millimetres at the borehole location.

4.3 Fill Material

Fill material was encountered below the topsoil layer.

The fill material is composed of grey-brown silty clay overlying brown sandy silt with some clay and traces roots.

The thickness of the fill layer is about 1.3 m and extends to a depth of approximately 1.4 m below ground surface at the borehole location (elevation (El.) 64.8 m).

Moisture content testing carried out on samples of the fill indicate a moisture content ranging between about 31 and 41 percent.

4.4 Former Topsoil Layer

An organic layer, anticipated to be the former topsoil layer, was encountered below the fill material at a depth of about 1.4 m below ground surface. The former topsoil layer consists of brown silty sand with trace clay and some roots. The former topsoil layer has a thickness of about 0.3 m and extends to a depth of approximately 1.7 m below ground surface (El. 64.5 m).

4.5 Clay

A native deposit of clay with some silt and trace sand was encountered below the fill materials at the borehole location.

The upper portion of the clay is grey-brown and considered to be weathered. The weathered clay crust has a thickness of about 2.0 m and extends to a depth of approximately 3.7 m below ground surface (El. 62.5 m).

Standard penetration tests carried out within the upper clay (weathered crust) gave N values of 1 to 3 blows per 0.3 metres of penetration, and based on the in situ shear vane strength results and our experience in similar local conditions, this indicates a stiff to very stiff consistency.

A grey clay was encountered below the weathered clay crust at a depth of approximately 3.7 m below ground surface (El. 62.5 m). Borehole 22-1 was terminated within the grey clay at a depth of approximately 25 m below ground surface (El. 41.20 m).

Standard penetration tests carried out in the grey clay gave N values of 1 to 2 blows per 0.3 metres of penetration. In situ vane shear strength tests carried out in the grey clay gave shear strength values ranging from 50 to 100 kilopascals, which indicates a stiff consistency. The sensitivity of the clay ranges between 5 and 14.

A continuous profile of the undrained shear strength in the clay was determined at the SCPT location using the following equation by Lunne et al (1997):

$$c_u = (Q_t - \sigma_v)/N_{kt}$$

Where;

c_u = Undrained shear strength (kilopascals)

Q_t = Cone Tip Stress (kilopascals)

σ_v = Total overburden pressure (kilopascals)

N_{kt} = Correction factor; assumed to be 11 in this case

The results of the SCPT data show that the average undrained shear strength of the clay steadily increased with depth from 2 m to 25 m below ground surface. For the purposes of the slope stability analysis, the average shear strength values with depth have been summarized in Table 4.1 below.

Table 4.1: Summary of Undrained Shear Strength with Depth

Depth Range (m)	Average Undrained Shear Strength (Kilopascals)	Consistency
0 to 4	50	Stiff

4 to 16	65	Stiff
16 to 23	80	Stiff
23 to 26	110	Very Stiff

The SCPT logs along with a shear strength plot determined from the SCPT data and corresponding field vane data are provided in Appendix C.

Grain size distribution tests were carried out on samples of the clay recovered from Borehole 22-1. The results are provided on the Soils Grading Chart in Appendix D and summarized in Table 4.2 below.

Table 4.2 – Summary of Grain Size Distribution Testing (Clay)

Location	Sample Number	Sample Depth (metres)	Gravel (%)	Sand (%)	Silt (%)	Clay (%)
22-1	4	2.3 – 2.9	0	1	10	89
	7	6.1 – 6.7	0	1	18	81
	10	21.3 – 21.9	0	1	16	83

*The above percentages have been rounded to better represent the soil composition

Atterberg limit tests were carried out on selected samples of the clay recovered from Borehole 22-1. The results are provided on the Plasticity Chart in Appendix D and are summarized in Table 4.3 below.

Table 4.3 – Summary of Atterberg Limits Testing (Clay)

Borehole	Sample Number	Depth (m)	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index	Moisture Content (%)
22-1	7	6.1 – 6.7	57.7	29.6	28.2	75.3
	10	21.3 – 21.9	62.5	24.7	37.8	70.0

The results show that the clay is of high plasticity.

Moisture content testing carried out on selected samples of the clay indicate a moisture content ranging between about 47 and 75 percent. The moisture content is generally above the liquid limit value.

The borehole was terminated within the grey silty clay at about 25 m below ground surface (El. 41.2 m).

4.6 Groundwater Levels

The groundwater level was measured in the monitoring well on June 27, 2022. At that time, the groundwater was located approximately 9.5 m from the ground surface (El. 56.7 m), and is not anticipated to represent the stabilized groundwater level within the clay unit. Based on the current site investigation work, the design groundwater level was taken to be at about 4 m below ground surface (El. 62.2 m).

It is anticipated that the groundwater level is influenced by the level of the adjacent watercourse at this site. The groundwater levels may be higher during wet periods of the year such as the early spring or following periods of precipitation.

4.7 Slope Geometry

A site reconnaissance was carried out by a member of our geotechnical engineering staff on April 20, 2022. At that time, the geometry of the slope at the site was measured using out Trimble R10 GPS surveying equipment, where possible, and manual surveying equipment. It should be noted that the slope along the south perimeter of the site is irregular, and signs of slope instability and erosion were noted in some areas of the slope (e.g., tension cracks, active erosion at the toe, etc.). For this study, the area of interest has been limited to the representative slope closest to the proposed works on site which is anticipated to have the highest potential influence to change in stability due to the proposed works.

The overall slope height and width (i.e., rise over run) as measured between the top and toe of the slope at the analyzed location is about 16 m to 19 m and about 47 m to 52 m (horizontal distance), respectively. A 6 m to 8 m wide meandering watercourse exists at the toe of the slope.

In general, the slope at the site is vegetated with grass and young to mature trees. At the location of the measured sections, no tension cracks were observed. Active surficial erosion and some mirror erosion along the banks of the watercourse were noted in the vicinity of the analyzed areas, but are not the subject of the current investigation.

5.0 SLOPE STABILITY ASSESSMENT

5.1 General

The purpose of the slope stability assessment undertaken is to estimate if the proposed works would impact the existing stability of the adjacent slopes at this site. The slope stability analysis was carried out using SLIDE, a state of the art, two dimensional limit equilibrium slope stability program using the Morgenstern-Price method at Sections A-A’ and B-B’. The sections were chosen to represent the potential “worst case scenario” in the area of the proposed work. The approximate locations of the cross sections considered is provided on the Borehole and Test Hole Location Plan, Figure A1. The results of the slope stability analyses are provided in Appendix E.

5.2 Input Parameters

The soil conditions used in the stability analysis were based on the subsurface conditions encountered in the borehole and SCPT hole advanced near the crest of the slope, observations from our site visit, and our experience with similar site conditions.

The slope stability analysis was carried out using clay strength parameters typical for the Ottawa area. To determine the existing factor of safety against overall rotational failure, the slope stability analyses were carried out using drained soil parameters, which reflect long-term conditions, and undrained soil parameters for the pseudo-static (i.e., seismic) condition. The summarizes soil parameters used in the analyses are provided in Table 5.1 below.

Table 5.1 - Summary of Soil Parameters

Soil Type	Effective Angle of Internal Friction, φ (degrees)	Effective Cohesion, c' (kPa)	Undrained Shear Strength, c_u (kPa)	Unit Weight, γ (kN/m^3)
Fill Material	34	1	-	18
Weathered Clay Crust	34	5	50	17.5
Grey Clay	32	7.5	65 to 110[1.]	17

Note: 1. See analysis considerations in Appendix E

To simulate traffic loading within the extent of the rehabilitation works (parking areas, access roadways), a uniform surcharge of 12 kilopascals was applied at ground surface.

The results of a stability analysis are highly dependent on the assumed groundwater conditions. We have modeled the groundwater level based on the level of the adjacent watercourse, the observed subsurface conditions, the level measured in the monitoring well installed on site, and the observed surface water noted on the north end of the site during our site visit at the time of our site visit. We have also assumed the bedrock surface to be located approximately 38 m below ground surface (considered to be a conservative estimate based on a water well record in the area).

5.3 Existing Factor of Safety

The slope stability analysis was carried out accounting for the soil parameters, anticipated groundwater conditions and approximate slope profile to represent the worse-case scenario for slope stability conditions. For the purposes of this study, a computed factor of safety of less than 1.0 to 1.3 for the long-term (static) condition is considered to represent a slope bordering on failure to marginally stable, respectively; a factor of safety of 1.3 to 1.5 is considered to indicate a slope that is less likely to fail in the long-term and provides a degree of confidence against failure ranging from marginal (1.3) to adequate (1.4 and greater) should conditions vary from the assumed conditions. A factor of safety of 1.5, or greater, is considered to indicate adequate long-term stability. For pseudo-static or seismic conditions, a factor of safety greater than 1.1 is considered acceptable.

The slope stability analysis indicates that the existing slope at Section A-A’, in its current configuration, has a factor of safety against overall rotational failure of approximately 1.5 for static loading conditions. At the limit of the proposed rehabilitation work, the factor of safety against overall rotational failure is about 2.3 (refer to Figure E1 in Appendix E).

The slope stability analysis indicates that the existing slope at Section B-B’, in its current configuration, has a factor of safety against overall rotational failure of approximately 1.5 for static loading conditions. At the limit of the proposed rehabilitation work, the factor of safety against overall rotational failure is about 2.1 (refer to Figure E5 in Appendix E).

Pseudo-static slope stability analyses were also carried out in an attempt to model the potential seismic loading conditions. A seismic coefficient (k_h) of 0.15 was used in the analysis (i.e., half of the Peak Ground Acceleration for the Ottawa area according to the Ontario Building Code 2015). The slope stability analyses indicate the slope, in its current configuration, has a factor of safety against instability of approximately 1.1 for pseudo-static (seismic) conditions, which is considered acceptable (refer to Figures E3 and E7 in Appendix E).

5.4 Influence of Proposed Rehabilitation Works on the Slope

To model the potential increase in loading associated with proposed development, a uniform surcharge of 3 kilopascals was added to the traffic loading (for a total of 15 kilopascals) at the ground surface along the entire length of the rehabilitation works to represent the proposed raising the pavement structure up to 100 millimetres.

Based on the results of the slope stability analyses, the findings indicate that the rehabilitation of the parking area for both Sections A-A’ and B-B’ will not alter the factor of safety of the slope (refer to Figures E2 and E6 in Appendix E) from the pre-construction conditions.

The slope was again analysed for pseudo static (seismic) conditions. Based on the results of the pseudo static analyses, the findings indicate that the rehabilitation of the parking area for both

Sections A-A’ and B-B’ will not alter the factor of safety of the slope (refer to Figures E4 and E8 in Appendix E) from the pre-construction conditions.

6.0 ADDITIONAL CONSIDERATIONS

Based on a review of available aerial photographs as well as our observations during the site reconnaissance, there is evidence of existing / ongoing erosion and potential instability issues of the slopes adjacent to the property within the ravine / creek area. In addition, other portions of the slopes which were not analyzed within the current scope of work may have a low factor of safety and could be prone to instability which may extend back into the upper table lands / flat areas. Further, slope instability at this site may occur which would not be related to the proposed works.

Based on communications received by the parties involved in the project, the purpose of the current geotechnical investigation is to estimate the impact, if any, of the proposed project on the existing stability of the slope. For further clarity, the purpose of the current slope stability study is not to assess the existing stability of the entirety of the slope along the length of the subject property, the potential impacts of the slope on the existing development on the property, or to provide rehabilitation options for the stabilisation of the slope (if required). Further, the current investigation is not intended to meet Infrastructure Ontario (IO) site investigation requirements for slope stability assessment. However, the above investigation results could be used to support a future IO slope stability investigation if undertaken by GEMTEC.

7.0 CLOSING

We trust this report is sufficient for your purposes. If you have any questions concerning this information or if we can be of further assistance to you on this project, please call.

Luc Bouchard

LICENSED PROFESSIONAL ENGINEER
L.R.J. BOUCHARD
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29 Jul 2022
PROVINCE OF ONTARIO

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Senior Geotechnical Engineer,

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Enclosures
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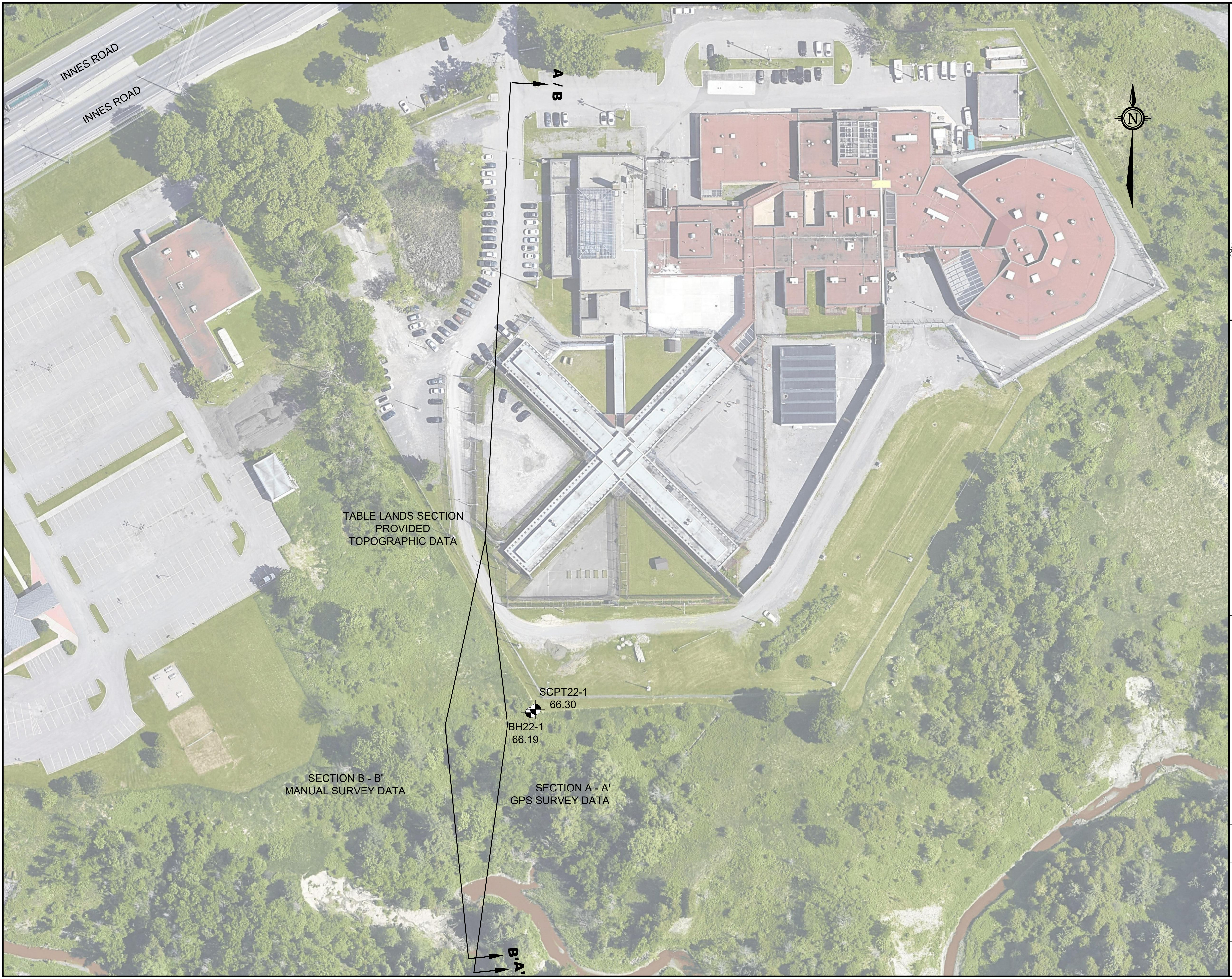
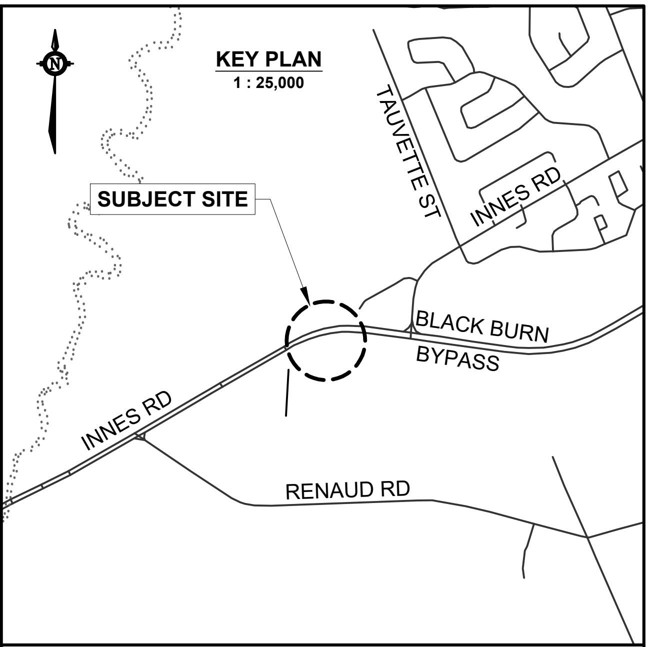
APPENDIX A

Borehole / Test Hole Location Plan, Figure A1





LEGEND

BH # — BOREHOLE ID
XX.XX — GROUND SURFACE ELEVATION, IN METRES GEODETIC DATUM

APPROXIMATE BOREHOLE / TEST HOLE LOCATION
(current investigation by GEMTEC)

A A'
SLOPE STABILITY ANALYSIS CROSS SECTION
(current investigation by GEMTEC)

Scale
1:1250
0 25 50 75m

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Drawing
BOREHOLE / TEST HOLE LOCATION PLAN

Client
OTTAWA CARLETON DETENTION CENTRE

Project
10000.037

2244 INNES ROAD
OTTAWA, ONTARIO

Drwn by S.L.
Chkd by L.B.

Date
JULY 2022

Rev.
0

FIGURE A1

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APPENDIX B

Record of Borehole

ABBREVIATIONS AND TERMINOLOGY USED ON RECORDS OF BOREHOLES AND TEST PITS

SAMPLE TYPES	
AS	Auger sample
CA	Casing sample
CS	Chunk sample
BS	Borros piston sample
GS	Grab sample
MS	Manual sample
RC	Rock core
SS	Split spoon sampler
ST	Slotted tube
TO	Thin-walled open shelby tube
TP	Thin-walled piston shelby tube
WS	Wash sample

SOIL TESTS	
w	Water content
PL, w_p	Plastic limit
LL, w_L	Liquid limit
C	Consolidation (oedometer) test
D_R	Relative density
DS	Direct shear test
G_S	Specific gravity
M	Sieve analysis for particle size
MH	Combined sieve and hydrometer (H) analysis
MPC	Modified Proctor compaction test
SPC	Standard Proctor compaction test
OC	Organic content test
UC	Unconfined compression test
γ	Unit weight

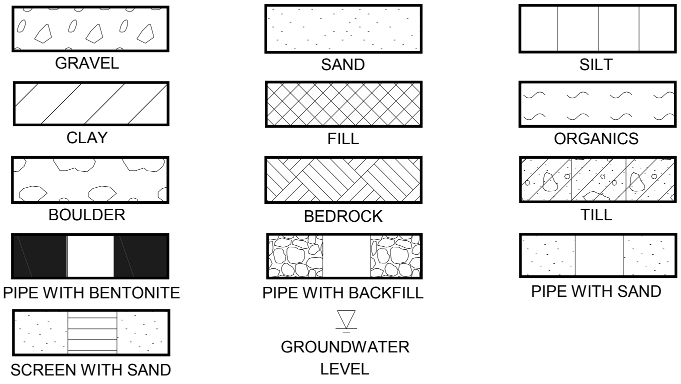
PENETRATION RESISTANCE

Standard Penetration Resistance, N
The number of blows by a 63.5 kg (140 lb) hammer dropped 760 millimetres (30 in.) required to drive a 50 mm split spoon sampler for a distance of 300 mm (12 in.). For split spoon samples where less than 300 mm of penetration was achieved, the number of blows is reported over the sampler penetration in mm.

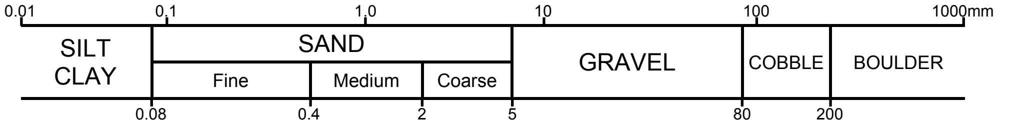
Dynamic Penetration Resistance
The number of blows by a 63.5 kg (140 lb) hammer dropped 760 mm (30 in.) to drive a 50 mm (2 in.) diameter 60° cone attached to ‘A’ size drill rods for a distance of 300 mm (12 in.).

WH	Sampler advanced by static weight of hammer and drill rods
WR	Sampler advanced by static weight of drill rods
PH	Sampler advanced by hydraulic pressure from drill rig
PM	Sampler advanced by manual pressure

COHESIONLESS SOIL Compactness		COHESIVE SOIL Consistency	
SPT N-Values	**Description**	**Cu, kPa**	**Description**
0-4	Very Loose	0-12	Very Soft
4-10	Loose	12-25	Soft
10-30	Compact	25-50	Firm
30-50	Dense	50-100	Stiff
>50	Very Dense	100-200	Very Stiff
		>200	Hard



GRAIN SIZE



DESCRIPTIVE TERMINOLOGY

(Based on the CANFEM 4th Edition)

0 10 20 35

TRACE	SOME	ADJECTIVE	noun > 35% and main fraction
trace clay, etc	some gravel, etc.	silty, etc.	sand and gravel, etc.

GEMTEC

descriptive terms.pub
Modified May 2018

RECORD OF BOREHOLE 22-1

CLIENT: Jp2g Consultants Inc.
PROJECT: 2244 Innes Road, Ottawa, Ontario
JOB#: 100009.037
LOCATION: See Test Hole / Borehole Location Plan, Figure A1

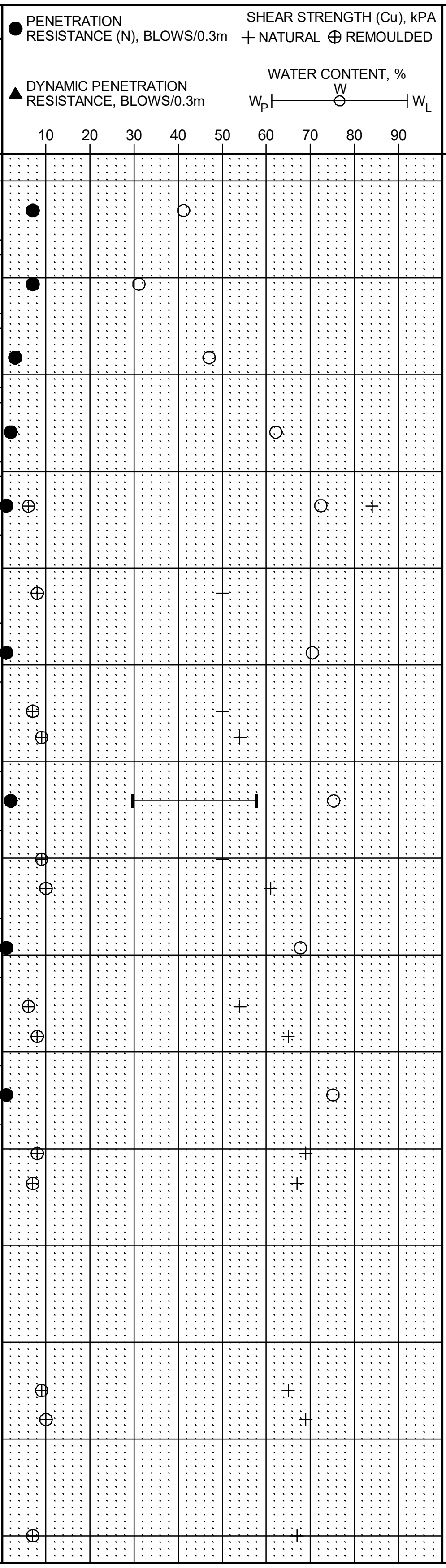
SHEET: 1 OF 2
DATUM: CGVD28
BORING DATE: Jun 15 2022

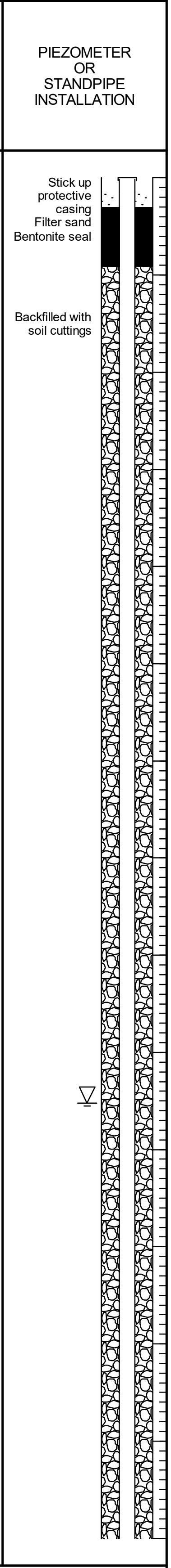
DEPTH SCALE METRES: 0, 1, 2, 3, 4, 5, 6, 7, 8, 9, 10, 11, 12, 13, 14

BORING METHOD: Power Auger, Hollow Stem Auger (210mm OD)

SOIL PROFILE		
DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)
Ground Surface		66.19
TOPSOIL		66.04 0.15
Grey brown, silty clay (FILL)		65.58 0.61
Brown sandy silt, some clay, trace roots (FILL)		64.82 1.37
Brown, silty sand, trace clay, some roots (FORMER TOPSOIL LAYER)		64.52 1.67
Very stiff to stiff, grey brown CLAY, some silt, trace sand (WEATHERED CRUST)		62.53 3.66
Stiff, grey CLAY, some silt, trace sand		

SAMPLES				
NUMBER	TYPE	RECOVERY, mm	BLOWS/0.3m	ADDITIONAL LAB. TESTING
1	SS	400	7	
2	SS	125	7	
3	SS	25	3	
4	SS	400	2	MH
5	SS	200	1	
6	SS	610	1	
7	SS	610	2	MH
8	SS	610	1	
9	SS	610	1	





GEMTEC
Consulting Engineers and Scientists

LOGGED: ML
CHECKED: LB

RECORD OF BOREHOLE 22-1

CLIENT: Jp2g Consultants Inc.
PROJECT: 2244 Innes Road, Ottawa, Ontario
JOB#: 100009.037
LOCATION: See Test Hole / Borehole Location Plan, Figure A1

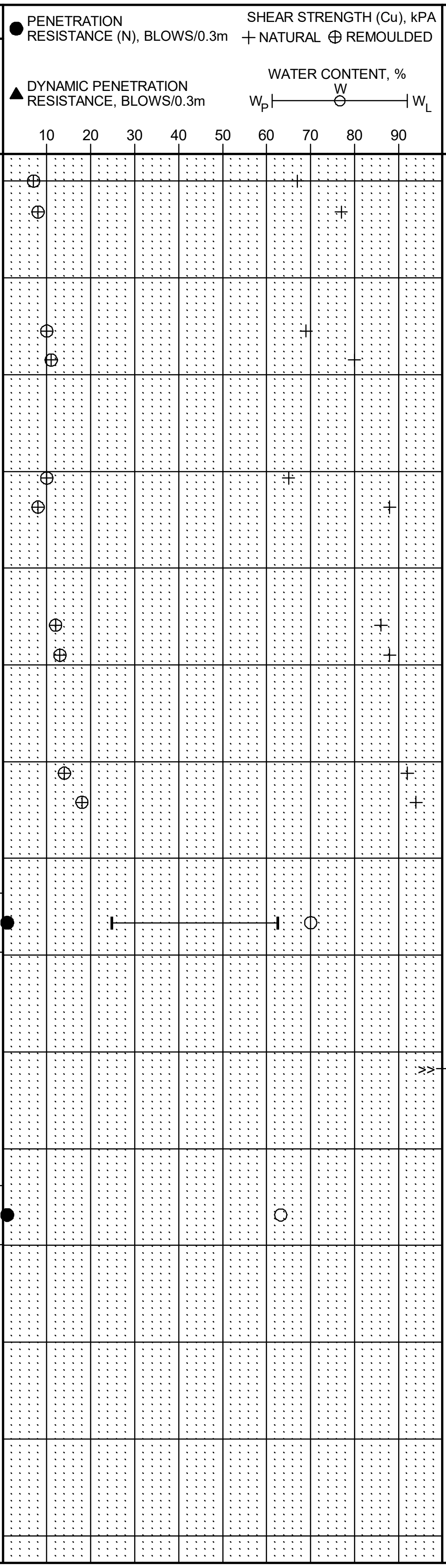
SHEET: 2 OF 2
DATUM: CGVD28
BORING DATE: Jun 15 2022

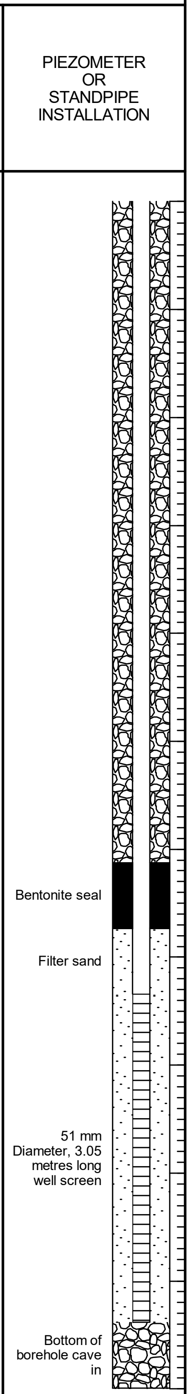
DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE			SAMPLES				ADDITIONAL LAB. TESTING
		DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	NUMBER	TYPE	RECOVERY, mm	BLOWS/0.3m	
14			▨						
21.6			▨		10	SS	610	1	MH
24.7			▨		11	SS	610	1	
25		End of Borehole		41.20 24.99					

Note:

1. Piezometer install as shown upon drilling completion.

2. Water level measured on June 27, 2022 as noted. Waterlevel is not anticipated to be representative of stabilized conditions.





GROUNDWATER OBSERVATIONS

DATE	DEPTH (m)	ELEV. (m)
22/06/27	9.5 ∇	56.7

GEMTEC
Consulting Engineers and Scientists

LOGGED: ML

CHECKED: LB

APPENDIX C

Piezocone Data

Report to: Jp2g Consultants Inc.
Project: 100009.037 R0 (July 29, 2022)

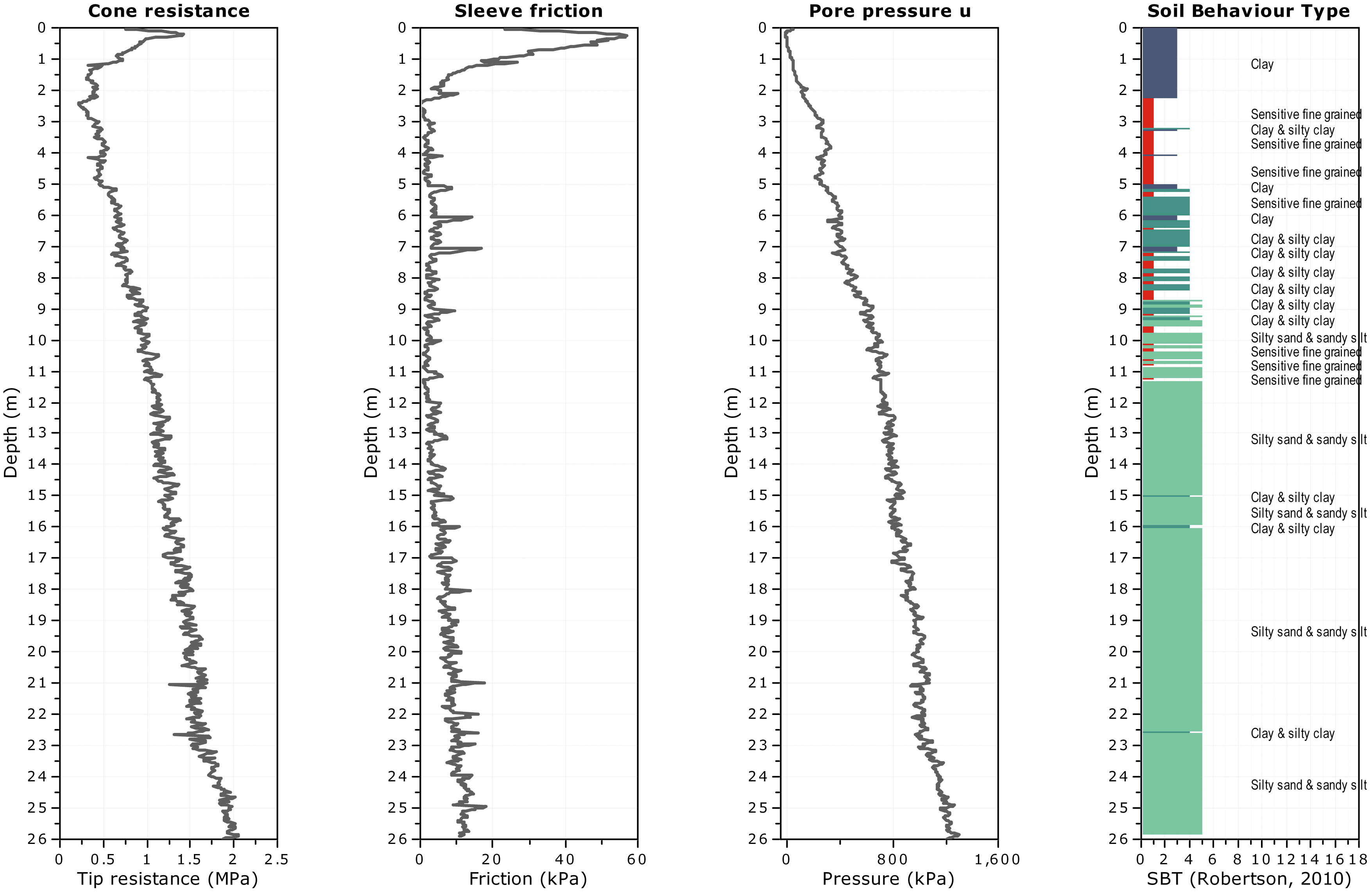
Figure C1: SCPT22-1

Project: **Slope Stability Assessment, Ottawa-Carleton Detention Centre, Parking Lot Expansion**

Location: **2244 Innes Road, Ottawa, Ontario**

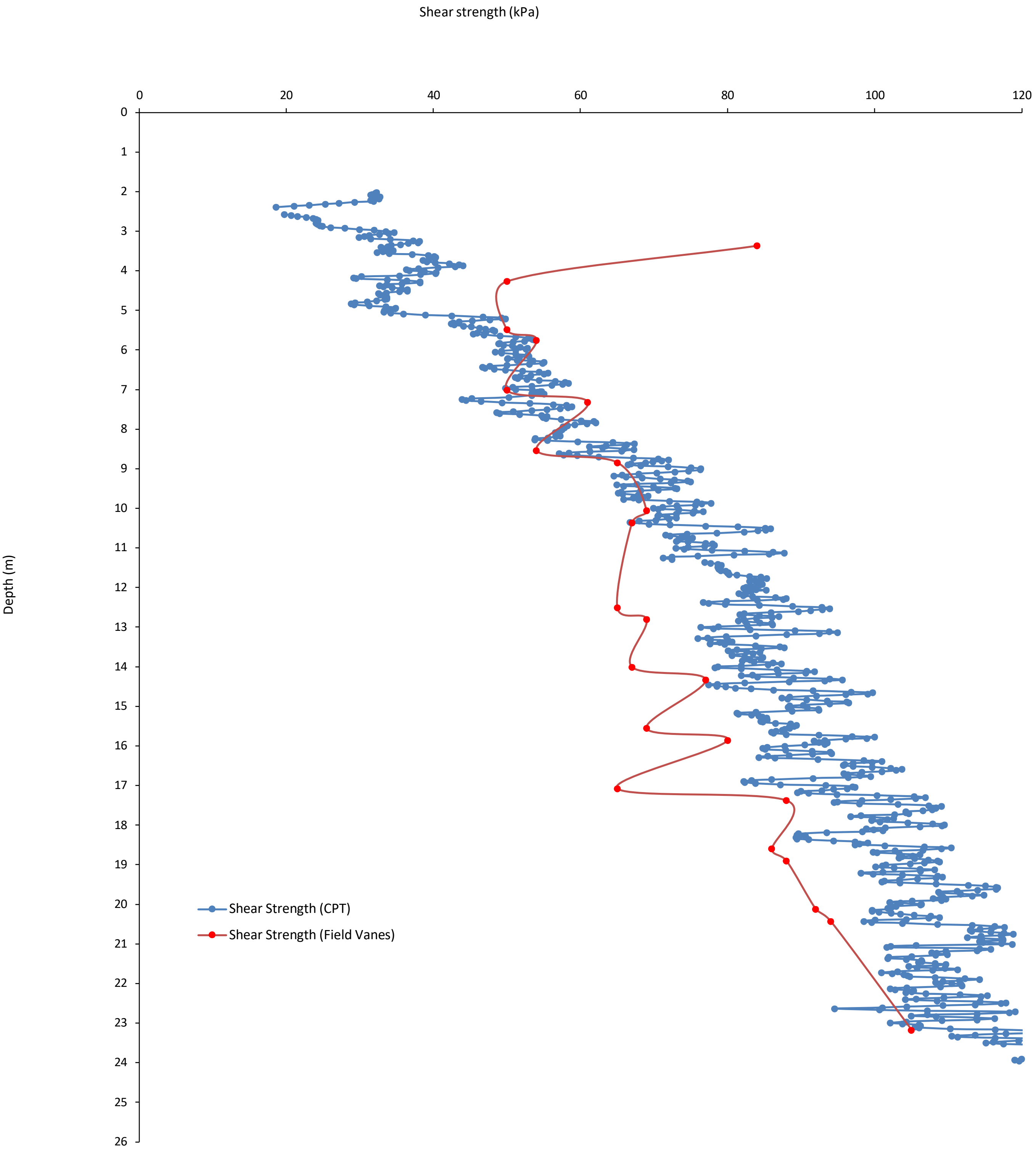
Total depth: 25.98 m, Date: 2022-06-16

Surface Elevation: 66.19 m



Summary of Undrained Shear Strength vs Elevation

FIGURE C2



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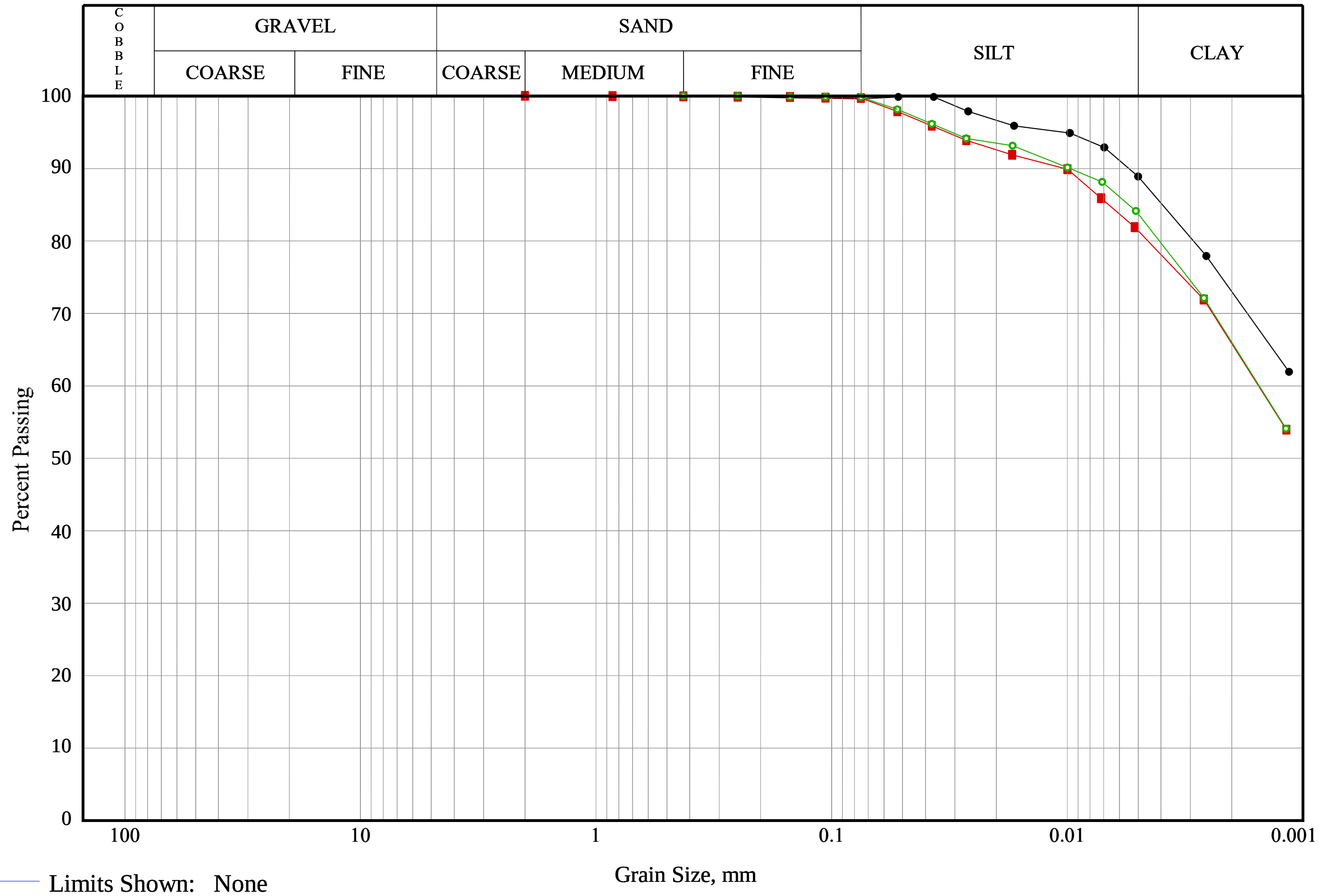
Date: July 2022
Project: 100009.037

APPENDIX D

Laboratory Testing Results

Report to: Jp2g Consultants Inc.
Project: 100009.037 R0 (July 29, 2022)

GEMTEC Consulting Engineers and Scientists	Client: Jp2g Consultants Inc. Project: 2244 Innes Road, Ottawa, Ontario Project #: 100009.037	Soils Grading Chart (LS-702/ ASTM D-422)



Limits Shown: None

Line Symbol	Sample	Borehole/ Test Pit	Sample Number	Depth	% Cob.+ Gravel	% Sand	% Silt	% Clay
●	Weathered Crust	22-1	04	2.3-2.9	0.0	0.4	10.7	88.9
■	Grey Clay	22-1	07	6.1-6.7	0.0	0.3	18.3	81.4
○	Grey Clay	22-1	10	21.3-21.9	0.0	0.2	16.1	83.7

Line Symbol	CanFEM Classification	USCS Symbol	D_{10}	D_{15}	D_{30}	D_{50}	D_{60}	D_{85}	% 5-75μm
●	Clay , some silt , trace sand	CH	---	---	---	---	---	0.0	10.7
■	Clay , some silt , trace sand	CH	---	---	---	---	0.0	0.01	18.3
○	Clay , some silt , trace sand	CH	---	---	---	---	0.0	0.01	16.1

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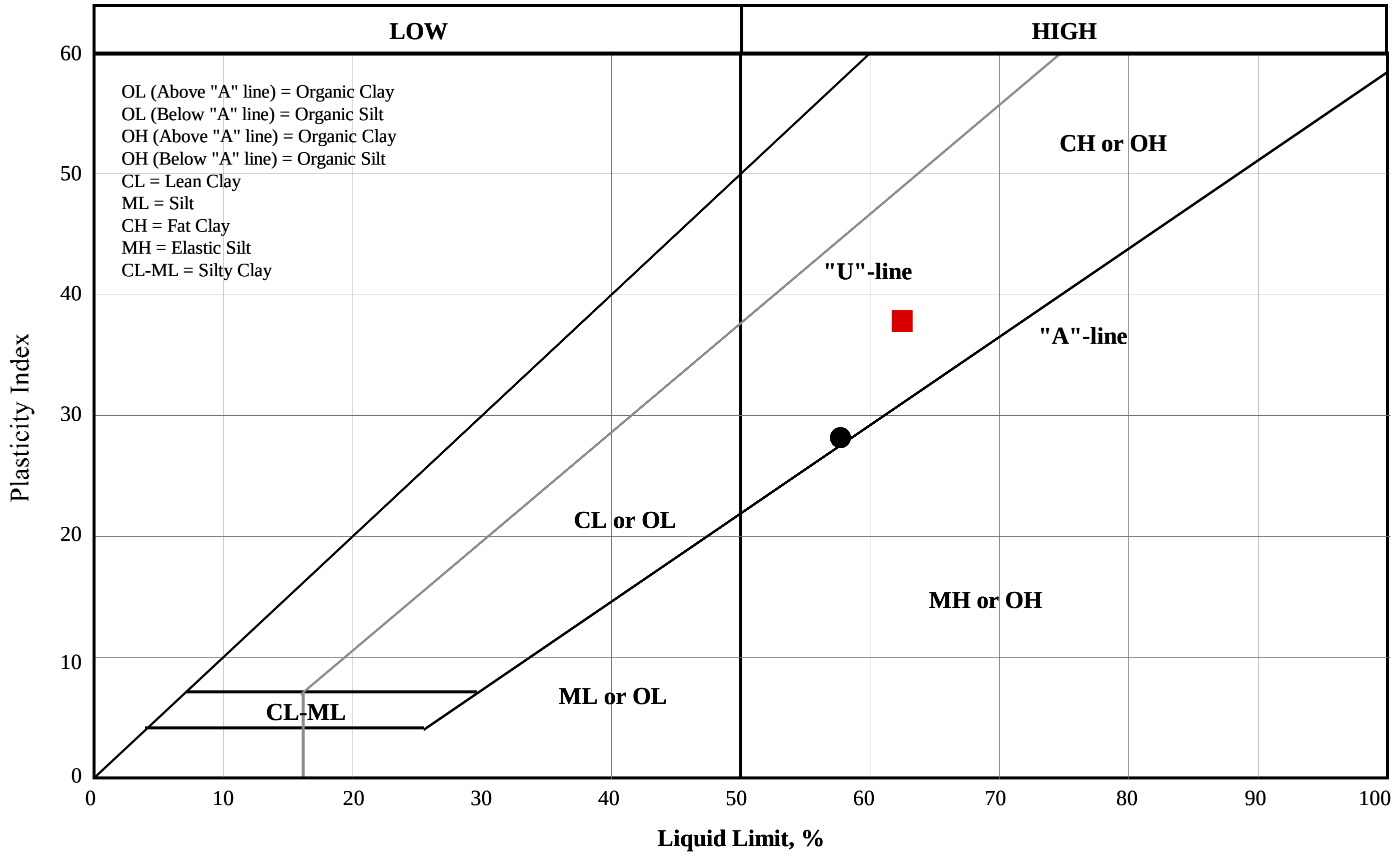
Client: Jp2g Consultants Inc.
Project: 2244 Innes Road, Ottawa, Ontario
Project #: 100009037

Plasticity Chart (D4318)



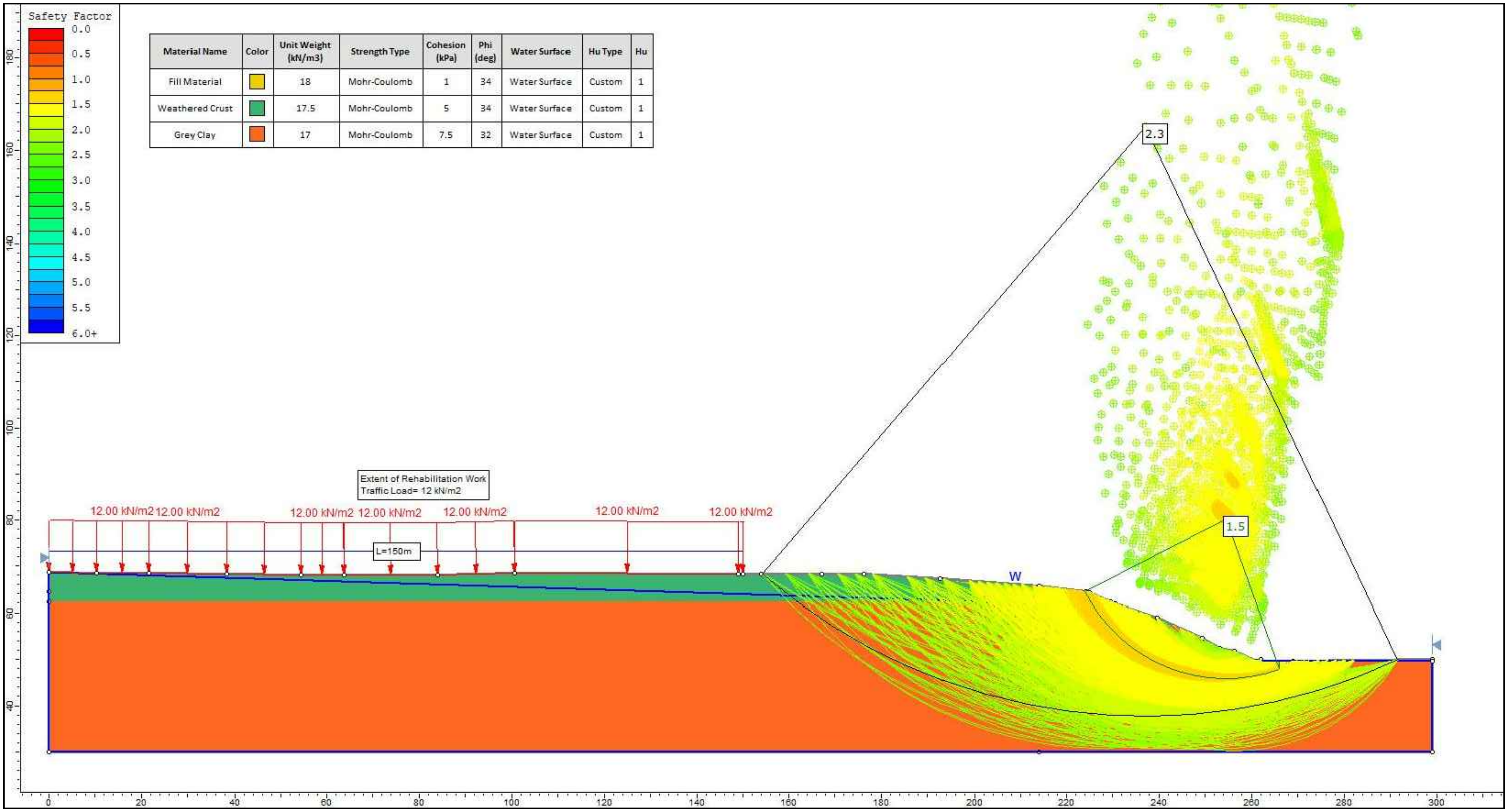
Symbol	Borehole /Test Pit	Sample Number	Depth	Liquid Limit	Plastic Limit	Plasticity Index	Non-Plastic	Moisture Content, %
●	22-1	07	6.1-6.7	57.7	29.6	28.2	☐	75.3
🟥	22-1	10	21.3-21.9	62.5	24.7	37.8	☐	70.0

Note: More information available upon request

APPENDIX E

Slope Stability Analysis Results

Report to: Jp2g Consultants Inc.
Project: 100009.037 R0 (July 29, 2022)



Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)	Water Surface	Hu Type	Hu
Fill Material	🟨	18	Mohr-Coulomb	1	34	Water Surface	Custom	1
Weathered Crust	🟩	17.5	Mohr-Coulomb	5	34	Water Surface	Custom	1
Grey Clay	🟧	17	Mohr-Coulomb	7.5	32	Water Surface	Custom	1

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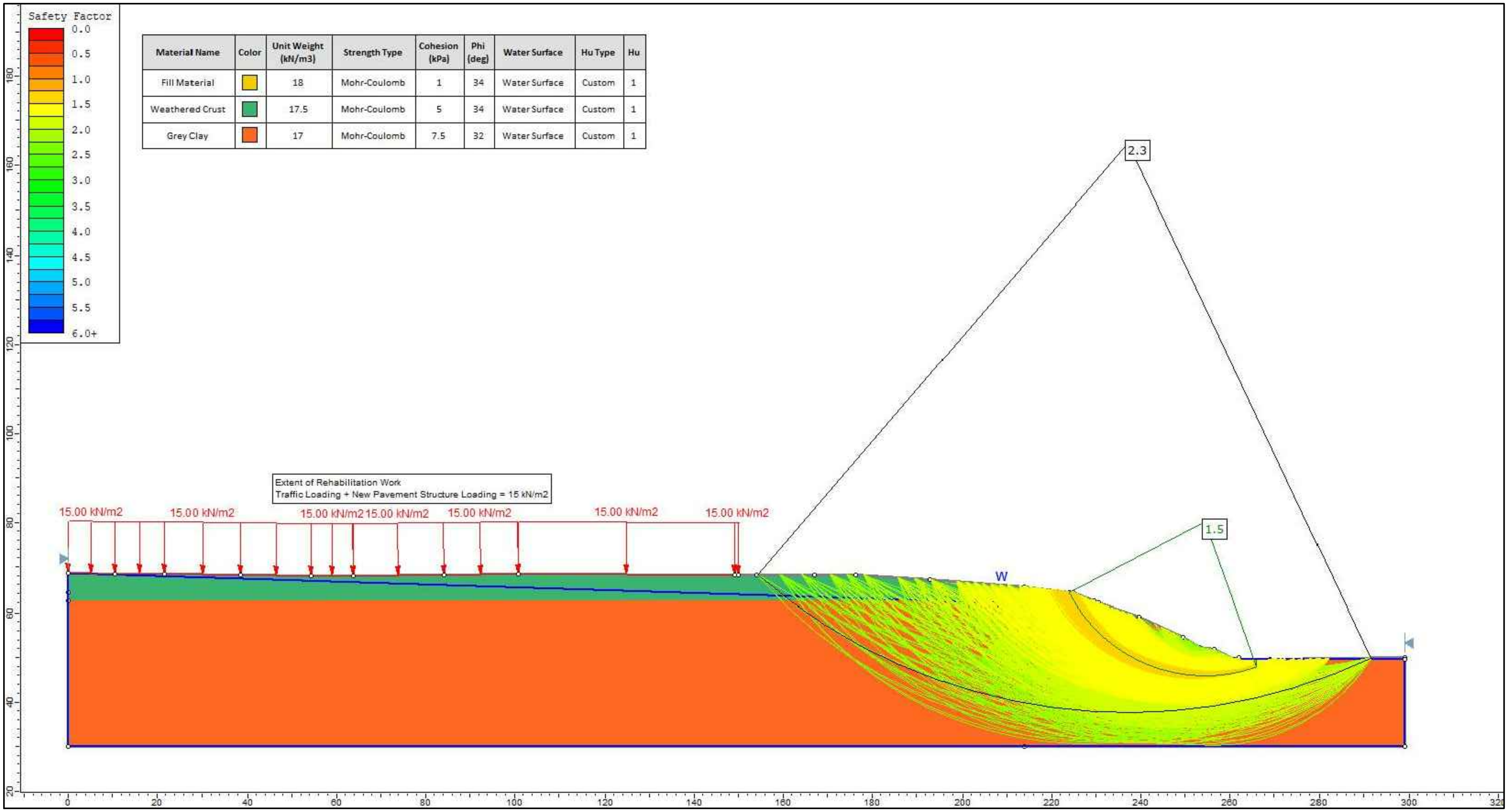
OTTAWA CARLETON DETENTION CENTRE
2244 INNES ROAD
OTTAWA, ONTARIO

FIGURE E1

File No.
100009.037

SECTION A-A' EXISTING CONDITION
DRAINED ANALYSIS



Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)	Water Surface	Hu Type	Hu
Fill Material	🟨	18	Mohr-Coulomb	1	34	Water Surface	Custom	1
Weathered Crust	🟩	17.5	Mohr-Coulomb	5	34	Water Surface	Custom	1
Grey Clay	🟧	17	Mohr-Coulomb	7.5	32	Water Surface	Custom	1

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OTTAWA CARLETON DETENTION CENTRE
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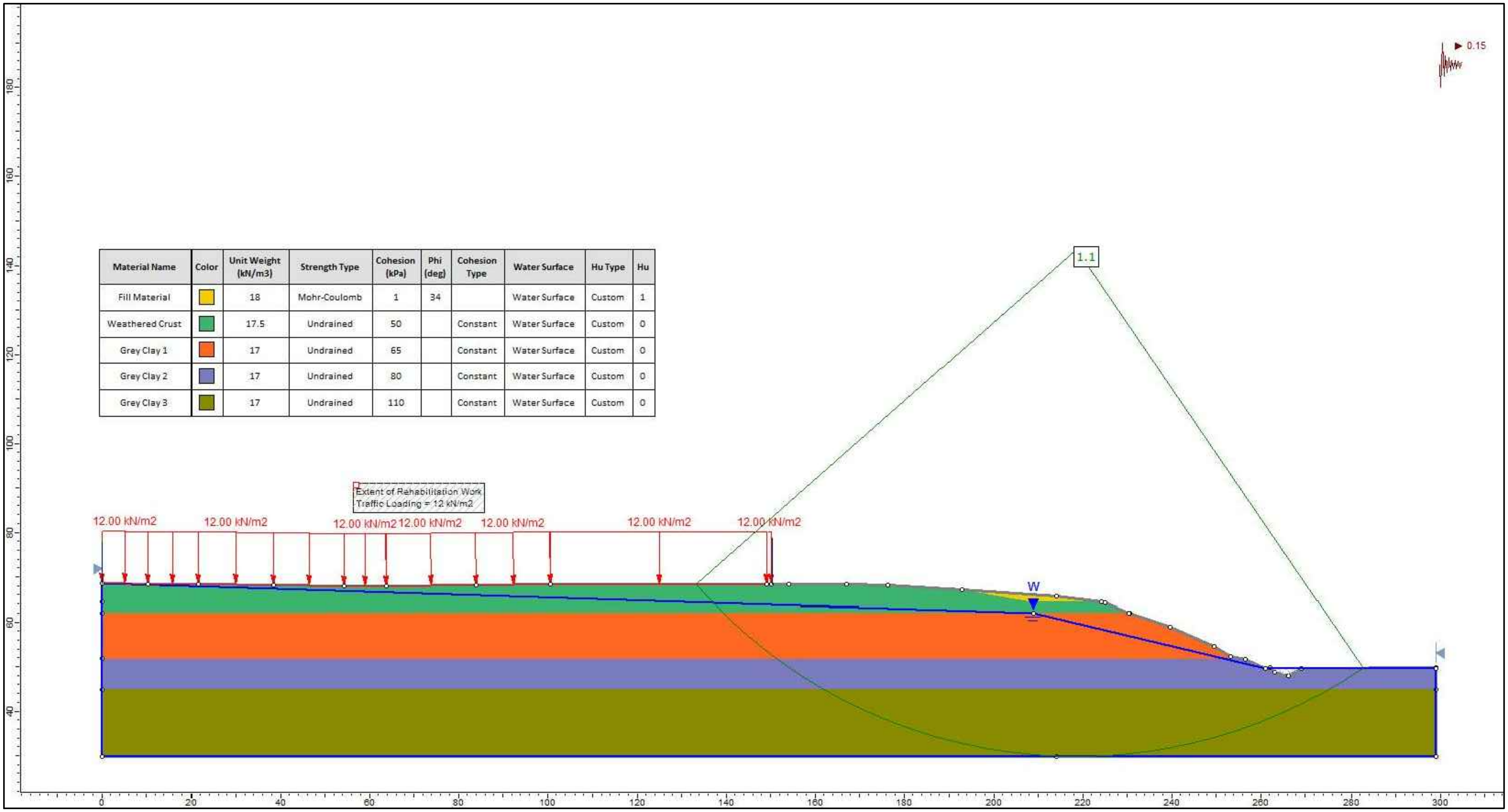
FIGURE E2

File No.
100009.037

SECTION A-A' PROPOSED REHABILITATION
DRAINED CONDITIONS



Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)	Cohesion Type	Water Surface	Hu Type	Hu
Fill Material	🟨	18	Mohr-Coulomb	1	34		Water Surface	Custom	1
Weathered Crust	🟩	17.5	Undrained	50		Constant	Water Surface	Custom	0
Grey Clay 1	🟧	17	Undrained	65		Constant	Water Surface	Custom	0
Grey Clay 2	🟪	17	Undrained	80		Constant	Water Surface	Custom	0
Grey Clay 3	🟫	17	Undrained	110		Constant	Water Surface	Custom	0

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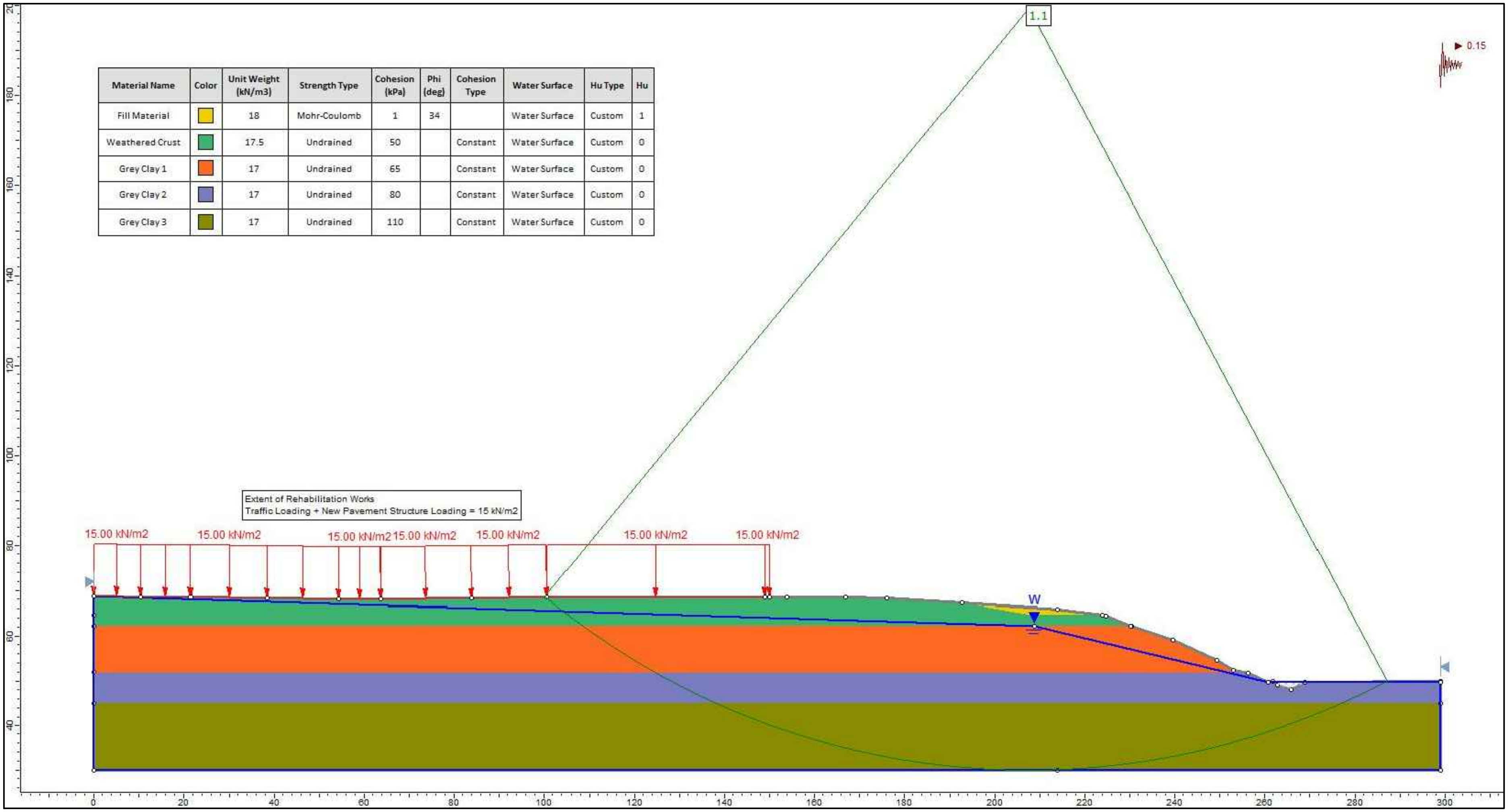
FIGURE E3

File No.
100009.037

SECTION A-A' EXISTING CONDITION
SEISMIC UNDRAINED ANALYSIS



Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)	Cohesion Type	Water Surface	Hu Type	Hu
Fill Material	🟨	18	Mohr-Coulomb	1	34		Water Surface	Custom	1
Weathered Crust	🟩	17.5	Undrained	50		Constant	Water Surface	Custom	0
Grey Clay 1	🟧	17	Undrained	65		Constant	Water Surface	Custom	0
Grey Clay 2	🟦	17	Undrained	80		Constant	Water Surface	Custom	0
Grey Clay 3	🟫	17	Undrained	110		Constant	Water Surface	Custom	0

Extent of Rehabilitation Works
Traffic Loading + New Pavement Structure Loading = 15 kN/m2

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OTTAWA CARLETON DETENTION CENTRE
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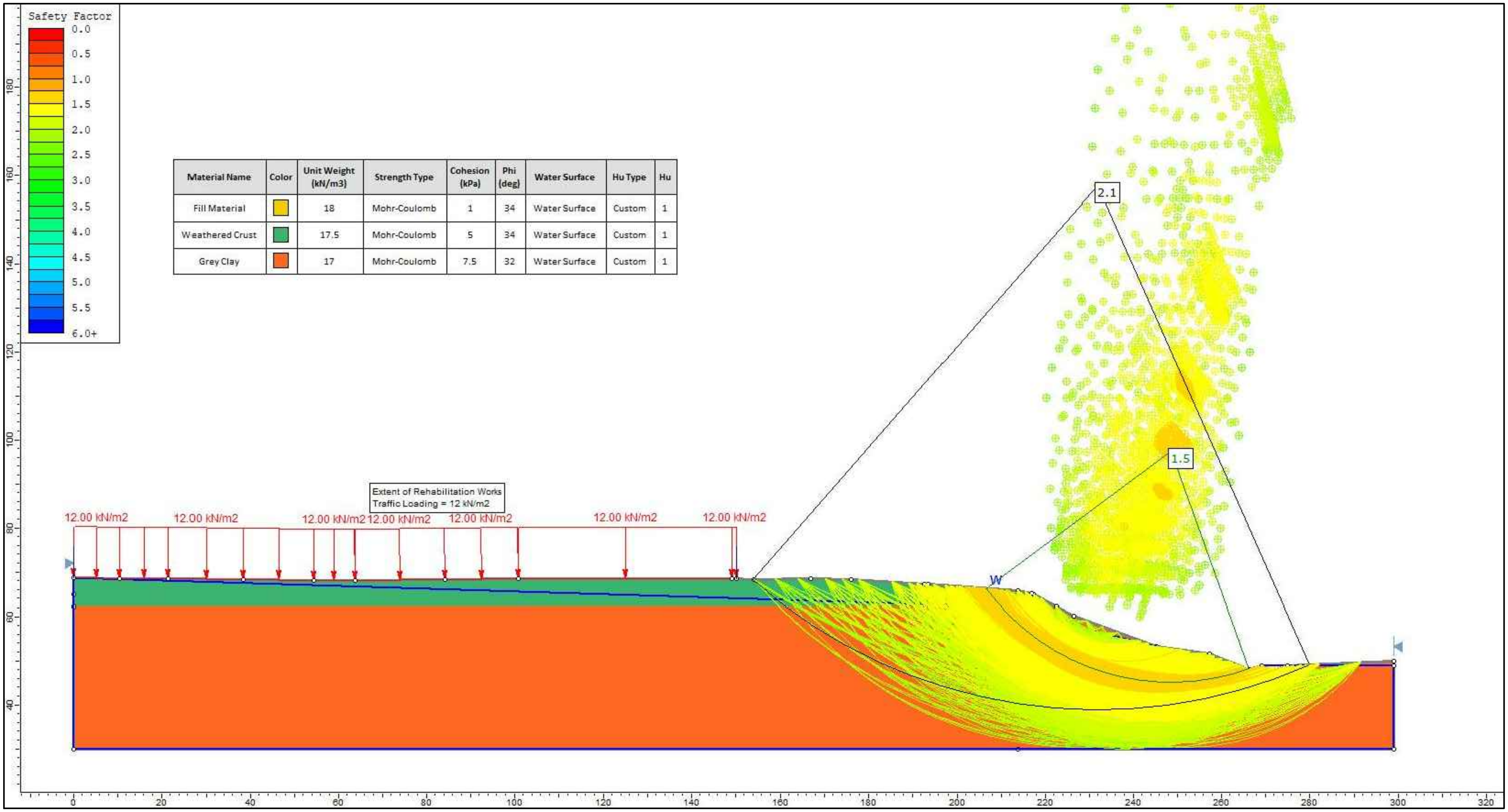
FIGURE E4

File No.
100009.037

SECTION A-A' PROPOSED REHABILITATION
SEISMIC UNDRAINED ANALYSIS



Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)	Water Surface	Hu Type	Hu
Fill Material	🟨	18	Mohr-Coulomb	1	34	Water Surface	Custom	1
Weathered Crust	🟩	17.5	Mohr-Coulomb	5	34	Water Surface	Custom	1
Grey Clay	🟧	17	Mohr-Coulomb	7.5	32	Water Surface	Custom	1

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OTTAWA CARLETON DETENTION CENTRE
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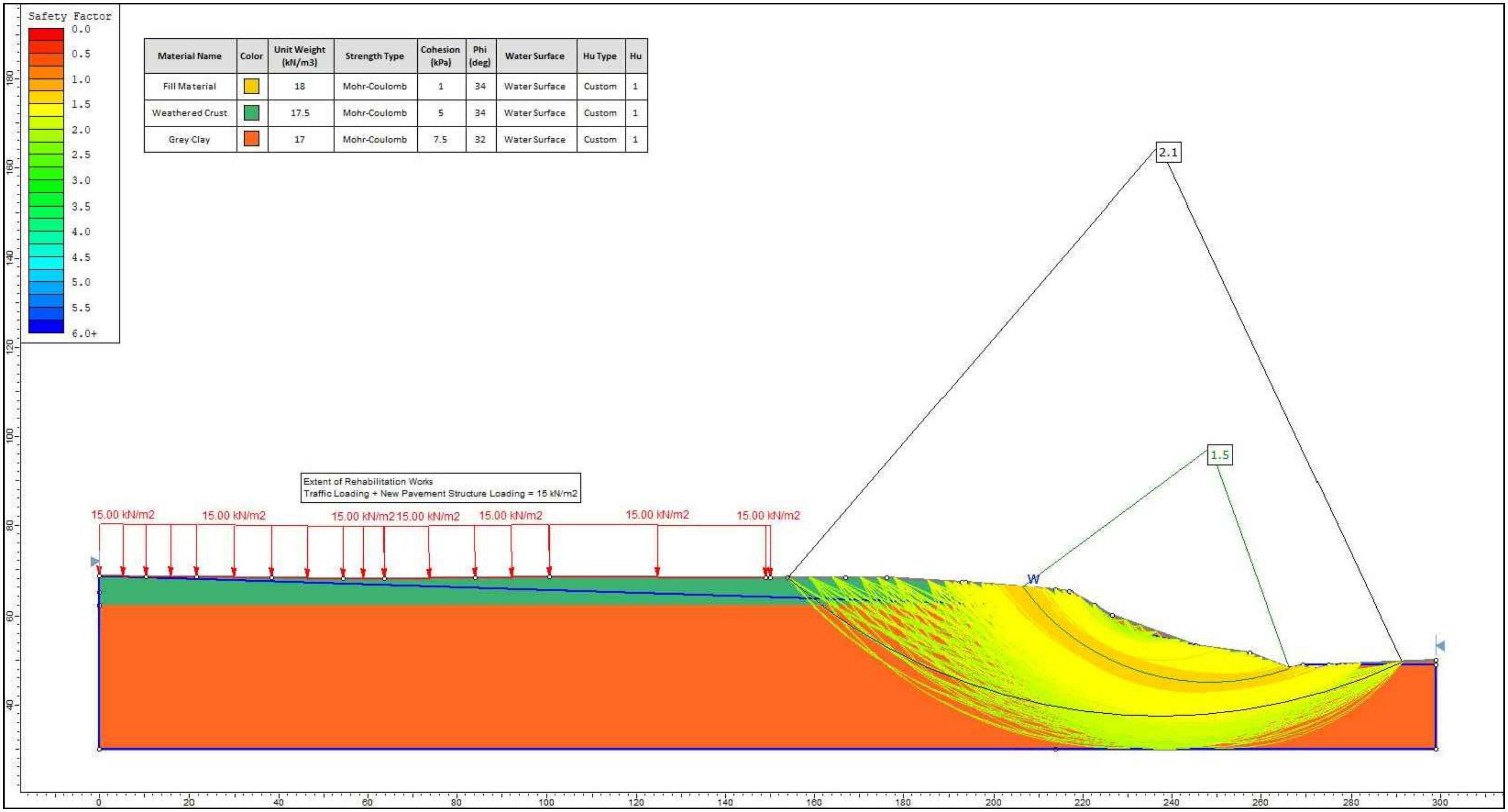
FIGURE E5

File No.
100009.037

SECTION B-B' EXISTING CONDITION
DRAINED ANALYSIS



Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)	Water Surface	Hu Type	Hu
Fill Material	🟨	18	Mohr-Coulomb	1	34	Water Surface	Custom	1
Weathered Crust	🟩	17.5	Mohr-Coulomb	5	34	Water Surface	Custom	1
Grey Clay	🟧	17	Mohr-Coulomb	7.5	32	Water Surface	Custom	1

Extent of Rehabilitation Works
Traffic Loading + New Pavement Structure Loading = 15 kN/m2

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OTTAWA CARLETON DETENTION CENTRE
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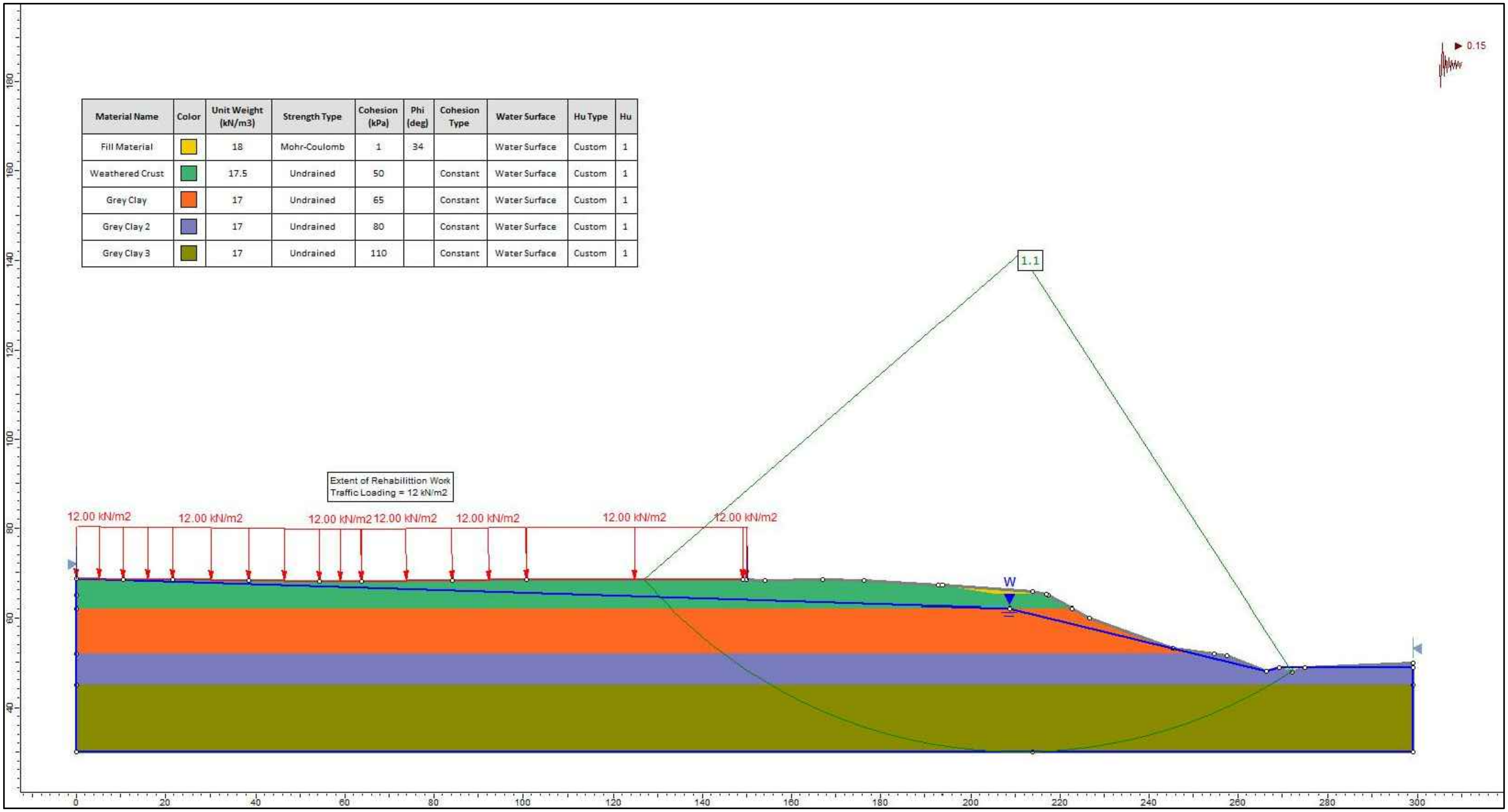
OTTAWA, ONTARIO

FIGURE E6

File No.
100009.037

SECTION B-B' PROPOSED REHABILITATION
DRAINED ANALYSIS



Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)	Cohesion Type	Water Surface	Hu Type	Hu
Fill Material	🟨	18	Mohr-Coulomb	1	34		Water Surface	Custom	1
Weathered Crust	🟩	17.5	Undrained	50		Constant	Water Surface	Custom	1
Grey Clay	🟧	17	Undrained	65		Constant	Water Surface	Custom	1
Grey Clay 2	🟪	17	Undrained	80		Constant	Water Surface	Custom	1
Grey Clay 3	🟫	17	Undrained	110		Constant	Water Surface	Custom	1

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OTTAWA CARLETON DETENTION CENTRE
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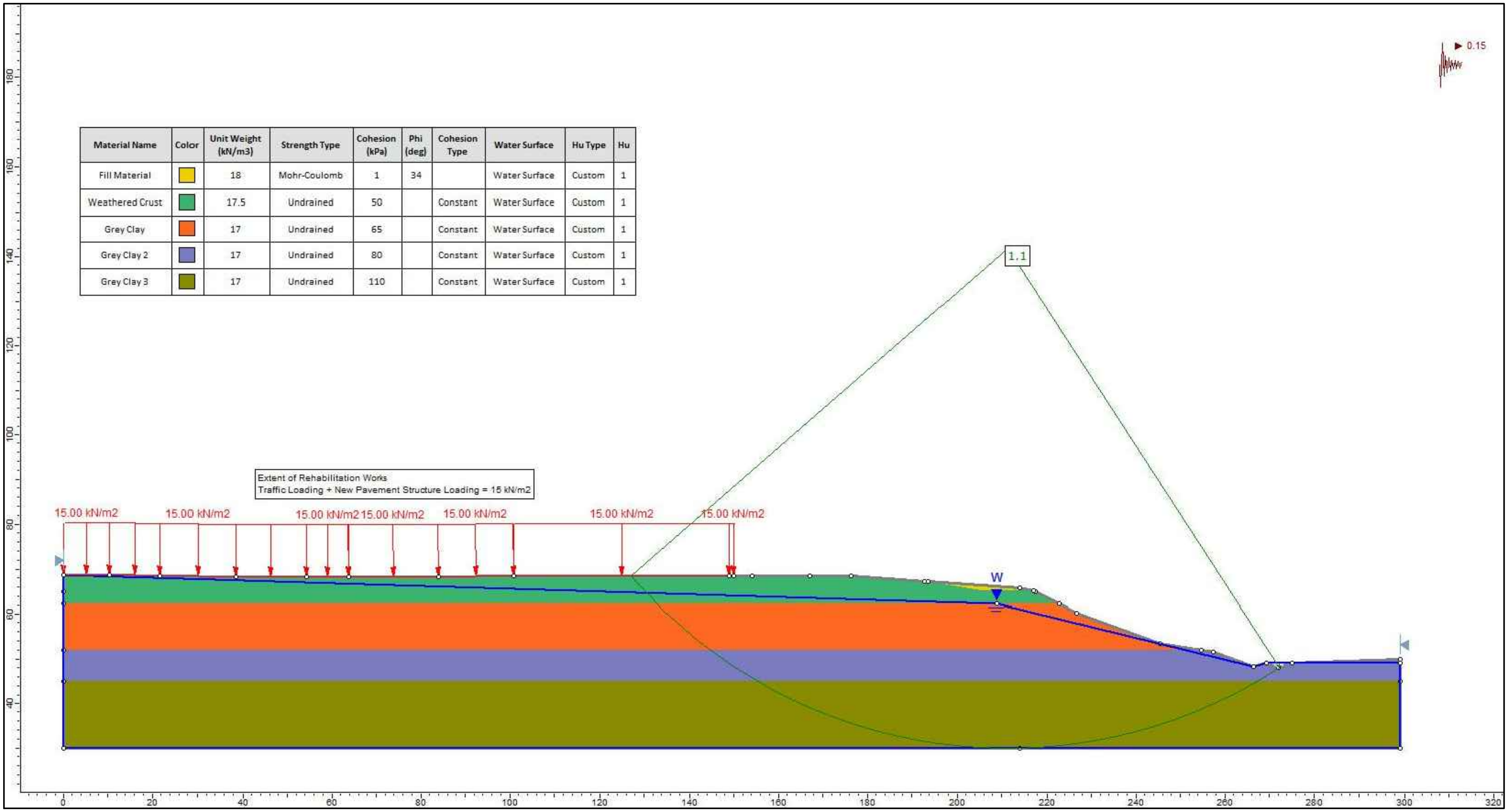
FIGURE E7

File No.
100009.037

SECTION B-B' EXISTING CONDITIONS
SEISMIC UNDRAINED ANALYSIS



Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)	Cohesion Type	Water Surface	Hu Type	Hu
Fill Material	🟨	18	Mohr-Coulomb	1	34		Water Surface	Custom	1
Weathered Crust	🟩	17.5	Undrained	50		Constant	Water Surface	Custom	1
Grey Clay	🟧	17	Undrained	65		Constant	Water Surface	Custom	1
Grey Clay 2	🟪	17	Undrained	80		Constant	Water Surface	Custom	1
Grey Clay 3	🟫	17	Undrained	110		Constant	Water Surface	Custom	1

Extent of Rehabilitation Works
Traffic Loading + New Pavement Structure Loading = 15 kN/m2

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OTTAWA CARLETON DETENTION CENTRE
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FIGURE E8

File No.
100009.037

SECTION B-B' PRPOSED REHABILITATION
SEISMIC UNDRAINED ANALYSIS

experience • knowledge • integrity

civil	civil
geotechnical	géotechnique
environmental	environnementale
field services	surveillance de chantier
materials testing	service de laboratoire des matériaux

expérience • connaissance • intégrité